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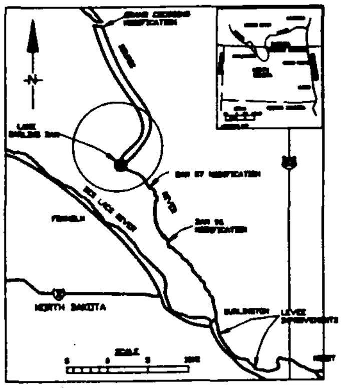

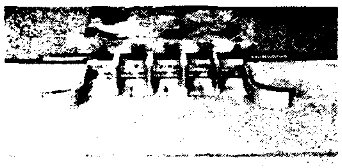

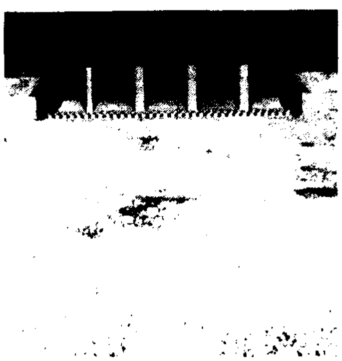

HYDRAULICS LABORATORY

TECHNICAL REPORT HL-88-9

LAKE DARLING SPILLWAY SOURIS RIVER, NORTH DAKOTA

Report 2

MODIFIED SPILLWAY

Hydraulic Model Investigation

by

Deborah R. Cooper

Hydraulics Laboratory

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13. ABSTRACT *(Maximum 200 words)*

The existing general spillway model of Lake Darling Dam was modified using the original linear scale ratio of 1:36. With modifications, the model reproduced the topography in an area extending 750 ft upstream, 1,700 ft downstream from the axis of the dam 485 ft to the left and 460 ft to the right of the center line of the spillway. Model analysis of its performance was desired to evaluate stilling basin action and riprap requirements and to obtain discharge characteristics of the relocated structure. This model investigation was particularly concerned with flow conditions in the approach and exit channels, spillway capacity, hydraulic performance of the stilling basin, and channel protective stone requirements. The hydraulic performance of the stilling basin was evaluated by conducting hydraulic jump "sweep out" tests to determine the factor of safety of the design for holding the jump in the basin by lowering the tailwater below the minimum expected tailwater elevation for various discharges. Velocities and center-line water-surface elevations were measured in the approach and exit channels for various conditions.

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PREFACE

The model investigation reported in the main report, Technical Report HL-88-9, was completed in September 1984. A "Risk Assessment Report" for Lake Darling Dam in November 1990 recommended several design changes that would impact on the operation and performance of the structure. A model study of these design changes was recommended.

The model investigation reported herein was authorized by the Office, Chief of Engineers, US Army, on 11 Feb 1991 at the request of the US Army Engineer District, St. Paul.

The studies were conducted in the Hydraulics Laboratory (HL) of the US Army Engineer Waterways Experiment Station (WES) during the period February 1991 to August 1991 under the direction of Messrs. F. A. Herrmann, Jr., Chief of the Hydraulics Laboratory, and G. Pickering, Chief of the Hydraulic Structures Division. The tests were conducted by Mrs. D. R. Cooper and Messrs. E. L. Jefferson and R. Bryant, Jr., of the Spillways and Channels Branch, under the direct supervision of Mr. N. R. Oswalt, Chief of the Spillways and Channels Branch. This report was prepared by Mrs. Cooper.

During the course of the investigation Messrs. G. Eggers of the St. Paul District (NCS) and J. Mazanec and H. Johnson of the North Central Division (NCD) visited WES to discuss test results and correlate these results with current design studies.

Mr. Lawrence Storey, Engineering and Construction Services Division, constructed the model.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

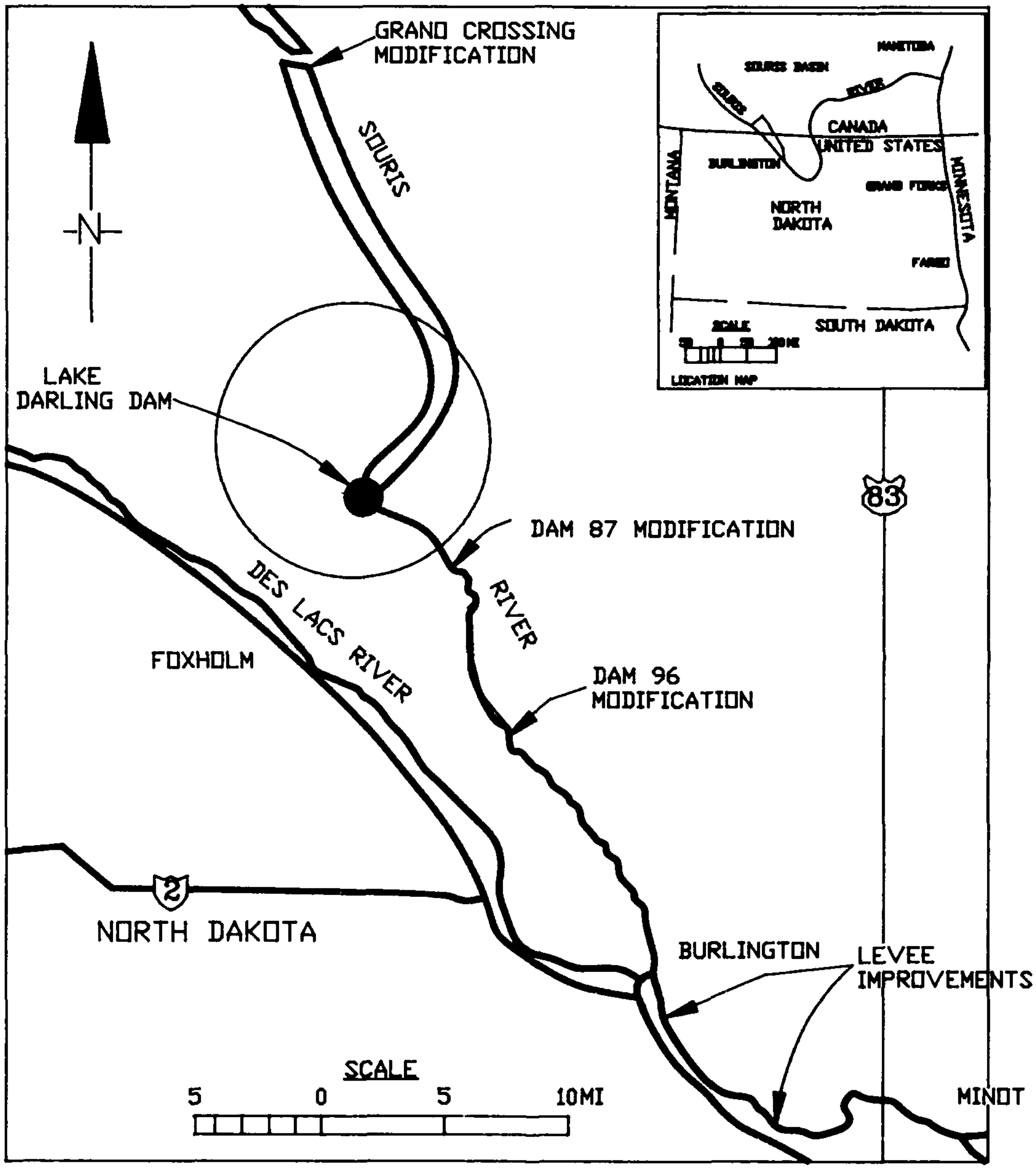

Figure 1. Location map

LAKE DARLING SPILLWAY

SOURIS RIVER, NORTH DAKOTA

MODIFIED SPILLWAY

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Lake Darling (Figure 1) is a large storage reservoir created by a dam northwest of Minot, ND, at the Ward-Renville County line on the Souris River (mile 429.9). The reservoir extends 27 miles* up a valley. The project is one unit in the Upper Souris National Wildlife Refuge and after modification will impound water for flood control and favorable waterfowl conditions downstream.

2. The initial model studies of Lake Darling spillway were completed in September 1984 and results of these tests are presented in the main report**. In 1987, the United States Government agreed to share the cost of building two reservoirs in Saskatchewan with the Canadian Government. The two reservoirs, Rafferty, on the Souris River upstream of Estevan, and Alameda, on Moose Mountain Creek upstream of Oxbow, will provide 100-year flood protection to the city of Minot. This international agreement greatly reduced the need to build a flood-control reservoir in North Dakota upstream of Minot, or even the need to raise Lake Darling Dam 4 ft as originally planned to provide Minot with additional flood protection. However, Lake Darling, as it now exists, is not a safe structure. The Lake Darling Spillway is inadequate to safely pass a large hypothetical flood on the order of the Standard Project Flood (SPF) or probable maximum flood (PMF); and the outlet works are not large enough to discharge the flows required under the new flood-control operation plan.

3. A "Risk Assessment Report" for Lake Darling Dam dated November 1990

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** Deborah R. Cooper. 1988 (Apr). "Lake Darling Spillway, Souris River, North Dakota; Hydraulic Model Investigation," Technical Report HL-88-9, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

recommended a spillway designed to pass 60,000 cfs with a maximum pool elevation of 1601.5.* The top of dam elevation will be lowered from 1614 (original design for new dam) to 1606. The spillway crest elevation will be at 1584 (as originally designed) and the crest length will be decreased from 255 to 251 ft which includes one 6-ft-wide pier and three 10-ft-wide piers. The spillway will have five tainter gates each 43 ft wide by 18 ft high. The structure will be located 750 ft into the valley from the right abutment based on recommendations from a US Fish and Wildlife value engineer proposal. The floor of the 51.5-ft-long hydraulic-jump-type stilling basin will be set at el 1575.0 and will contain two rows of 3.5-ft-high baffle piers and a 2-ft-high sloped end sill.

4. The modified dam will consist of a concrete gravity-type spillway structure flanked by compacted earth-fill embankments to high ground on the east and west sides of the river. The general plan and profile of the portion of the dam investigated in this phase of tests with model limits are shown in Plates 1 and 2.

Purpose and Scope of the Model Study

5. Although the modified design of Lake Darling Spillway was based on sound hydraulic design practice, physical model tests were desired to evaluate stilling basin action and riprap requirements and to obtain discharge characteristics of the modified and relocated structure as given in paragraphs 3 and 4. This model investigation was particularly concerned with (a) flow conditions in the approach and exit channels, (b) spillway capacity, (c) hydraulic performance of the stilling basin, and (d) channel protective stone requirements.

Presentation of Data

6. In the presentation of test results, no attempt is made to introduce the data in the chronological order in which the tests were conducted on the model. Instead, as each element of the structure is considered, all tests conducted thereon are discussed in detail. All model data are presented in terms of prototype equivalents. All tests are discussed in Part III.

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* All elevations (el) cited herein are in feet referenced to the National Geodetic Vertical Datum (NGVD).

PART II: THE MODEL

Description

7. The existing general spillway model of Lake Darling Dam was modified using the original linear scale ratio of 1:36. With modifications, the model reproduced the topography in an area extending 750 ft upstream, 1,700 ft downstream from the axis of the dam 485 ft to the left and 460 ft to the right of the center line of the spillway (Figure 2, Plate 1). The portions of the model representing the approach, exit, and overbank were molded in pea gravel. The channel and overbank were grouted with a fine dusting of cement. The spillway, spillway gates, and piers were constructed of sheet metal. The stilling basin, basin elements, and sidewalls were made of water-repellant-treated wood. The sluices were not reproduced in this phase of testing.

Appurtenances and Instrumentation

8. Water used in the operation of the model was supplied by pumps, and discharges were measured with venturi meters. The tailwater in the downstream end of the model was controlled by an adjustable tailgate. Steel rails set to grade provided reference planes. Water-surface elevations were obtained with point gages. Velocities were measured with an electromagnetic velocity meter. Current patterns were determined with the movement of dye injected into the water and confetti sprinkled on the water surface.

Scale Relations

9. The accepted equations of similitude, based upon the Froudian relations, were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and the prototype. General relations for the transference of model data to prototype equivalents are presented in the following tabulation:

Dimension	Ratio	Scale Relation
Length	$L_r = L$	1:36
Area	$A_r = L_r^2$	1:1,296

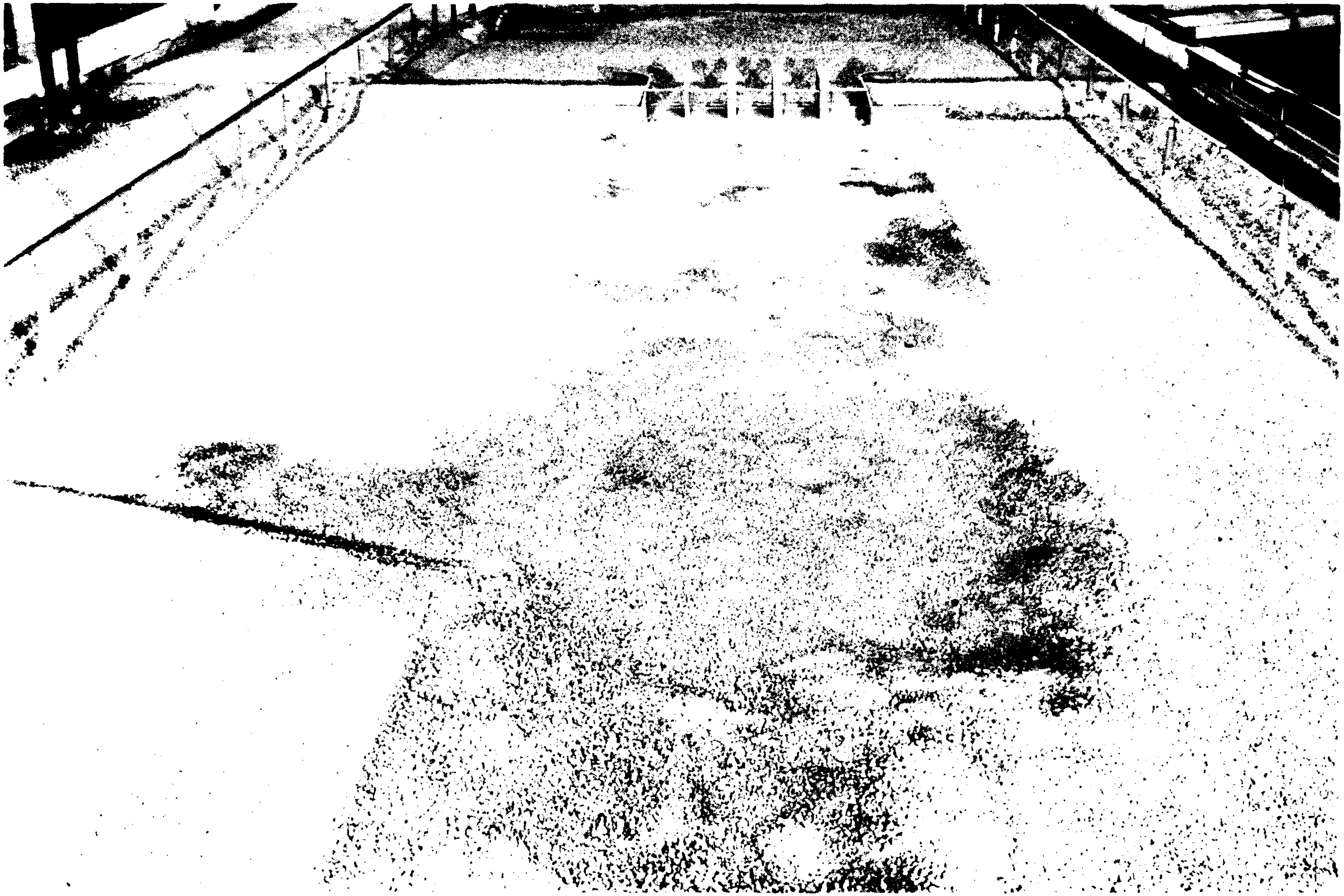

Figure 2. Type 3 design 1:36-scale model

Dimension	Ratio	Scale Relation
Weight	$W_r = L_r^3$	1:46,656
Velocity	$V_r = L_r^{1/2}$	1:6
Discharge	$Q_r = L_r^{5/2}$	1:7,776
Time	$T_r = L_r^{1/2}$	1:6

NOTE: Dimensions are in terms of length.

10. Quantitative measurements of discharge, water-surface elevation, time, and velocity in the model were converted to prototype dimensions by means of these scale relations. Experimental data also indicate that the prototype-to-model scale ratio is valid for scaling riprap in the sizes used in this investigation.

PART III: TESTS AND RESULTS

Approach Area

11. The general pattern of flow was much improved with the relocation of the spillway 750 ft out into the valley as opposed to the original abutment alignment for the 4-ft raise option that was previously model tested in 1984. The flatter, higher contours and elevated approach channel provided a shallower approach depth than with the original design. Flow conditions in the approach were satisfactory except in the immediate vicinity of the wing walls for discharges greater than 30,000 cfs. At these discharges, drawdown of the water surface around the wing walls was observed where lateral flow along the embankment was intercepted by the approach flows to the structure. No remedial measures were explored in the model because observations of flow patterns and velocity distribution across the structure indicated that the drawdown did not affect the hydraulic performance of the structure but was more of an aesthetic problem. This will be addressed later.

Spillway Crest and Gate Piers

12. Details of the spillway crest and gate piers are shown in Plates 2 and 3. The weir was 8.5 ft high with a 1-on-1 sloping upstream face with an ogee crest. The 6-ft- and 10-ft-wide piers had type 3 pier noses (HDC chart 111-5*) and extended 59.5 ft upstream from the weir crest. The piers terminated at the downstream toe of the crest. During model tests the St. Paul District considered modifying the shape of the pier noses to a shorter type 2 pier nose (HDC chart 111-5). Surging occurred upstream of the gates with the type 3 pier nose. After observation of the operation of the model, engineers from the St. Paul District decided that modification of the pier nose would create undesirable flow conditions near the gates with and without ice. Therefore, during the model tests, no changes were made to either the spillway weir or the crest piers.

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* US Army Corps of Engineers. Hydraulic Design Chart 111-5, Hydraulic Design Criteria, prepared for Office, Chief of Engineers, by US Army Engineer Waterways Experiment Station, Vicksburg, MS, issued serially since 1952.

Spillway Capacity

13. Spillway rating curves for all five gates at full and partial openings are shown in Plate 4 for free-flow conditions. Gate openings are referenced to the gate seat. The basic calibration data are shown in plots of discharge versus the approach channel energy elevation (water surface plus velocity head based on average velocity) on the weir for free flows at full and partial gates openings in Plates 5 and 6. Data used to plot these curves are shown in Table 1. These curves were obtained by introducing several constant discharges into the model for each gate opening and recording the corresponding upper pool elevation for minimum tailwater conditions. The water surface elevation was measured with a point gage located 700 ft upstream of the dam axis. The equation for each curve is the best empirical fit of the free-flow data by the method of least squares.

14. The spillway design was based on the assumption that the spillway would discharge about 58,000 cfs at maximum surcharge pool el 1601.0 and the 1984 model test results from the 4-ft-raise effort indicated the spillway would discharge about 58,000 cfs at pool el 1601.0.* Model results for this design, however, indicated that the spillway will only discharge about 53,000 cfs at pool el 1601.0.

15. The invert of the approach channel was lowered 5 ft (to el 1570.5) and the abutment radius was changed to 75 ft as shown in Plates 7 and 8 to determine if a lower channel invert would affect the upper pool elevation. The net effect of these changes was the lowering of the surcharge pool elevation from 1601.82 to 1601.63 for a discharge of 60,000 cfs and a minimum tailwater elevation (el 1592.5). Model tests also indicated a spillway capacity of 53,000 cfs at pool el 1601.0 and a minimum tailwater at el 1573.0 with the lower invert.

16. The tailwater was lowered in 1-ft increments to determine what effect, if any, tailwater submergence had on the surcharge pool. The data from these tests indicated that tailwater submergence accounts for less than 0.1 ft of the increased surcharge (Table 2).

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* Cooper, op. cit.

Type 6 Design Stilling Basin

17. The hydraulic performance of types 1-5 design stilling basins was discussed in the main report, TR-HL-88-9. The type 6 design stilling basin (Plate 3) consisted of a 51.1-ft-long apron at el 1575 with two rows of 3.5-ft-high baffle piers and a 2-ft-high sloping end sill. Sidewalls were vertical and were 18.0 ft high (top el 1595.0). With the basin raised to el 1575, the jump was held in the stilling basin for all discharges up to and including 60,000 cfs at minimum tailwater elevations. Tailwater elevations were set according to the expected tailwater curve shown in Plate 9.

Sweep out tests

18. Observation of flow conditions in the type 6 design basin revealed a marginal jump in the basin with resulting wave action in the exit area for the 60,000-cfs discharge. The hydraulic performance of the stilling basin was evaluated by conducting hydraulic jump "sweep out" tests. These tests were run to determine the factor of safety of the design for holding the jump by lowering the tailwater below the minimum expected tailwater elevation for 10,000, 30,000, and 60,000 cfs, respectively. The minimum tailwater elevation was set for each corresponding discharge, the upper pool was allowed to stabilize, and then the tailwater was dropped in 1-ft increments. When the hydraulic jump began to "sweep out" of the stilling basin, the corresponding tailwater elevation was recorded. The results of these tests are given in Table 3.

Velocities and water-surface profiles

19. Velocities and water-surface profiles were obtained. Velocities were measured in the exit channel with the minimum expected tailwater elevations in the model for 5,000 cfs with all gates open 1.7 ft and with one gate open full. Flow patterns and velocities 1 ft above the channel floor were plotted and are shown in Plates 10 and 11, respectively. Velocities and center-line water-surface elevations were measured in the exit channel with the minimum expected tailwater elevations in the model for 10,000 and 30,000 cfs. Flow patterns and velocities 1 ft above the channel floor were plotted and are shown in Plates 12 and 13, respectively. Water-surface profiles along the center line of the channel are plotted in Plates 14 and 15, respectively. Velocities and center-line water-surface elevations were

measured in the approach and exit channels with the minimum expected tailwater elevations in the model for 60,000 cfs. Flow patterns and velocities 1 ft above the channel floor were plotted and are shown in Plate 16. A water-surface profile along the center line of the channel is plotted in Plate 17.

Downstream wing walls

20. The right wing wall adjacent to the end sill was eliminated and riprap protection was wrapped around the right sidewall (Plate 18). Eliminating the right wing wall had no adverse impact on flow conditions.

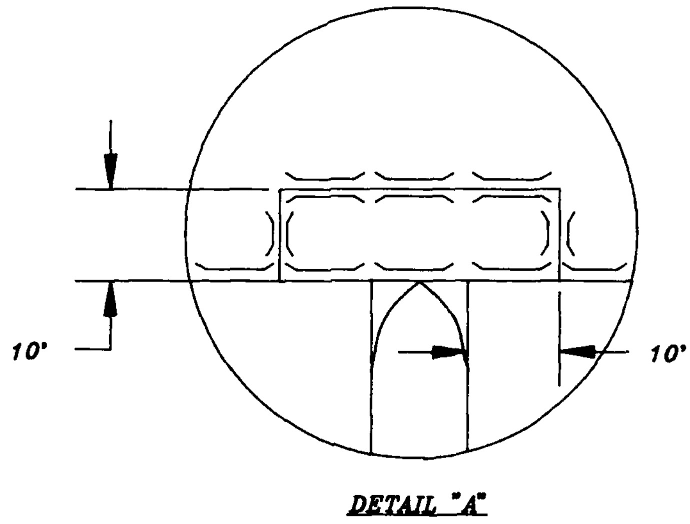
Riprap and Stone Protection Requirements

Upstream

21. Details of the rounded field stone protection in the approach area as tested in the model are shown in Plate 19. Initially, the approach area was covered with 100 ft (sta 0+59B to sta 1+59B) of protective round stone simulating prototype field stone with an average weight of 199 lb then 171 ft (sta 1+59B to sta 3+30B) of protective round stone simulating prototype field stone with an average weight of 35 lb. Gradation curves for all stone and riprap used in testing are plotted in Plates 20-23. The upstream stone protection remained stable for discharges up to and including 30,000 cfs. Some of the smaller stone around the pier noses was displaced at the 60,000-cfs discharge. Protective stone simulating prototype field stone with an average weight of 283 lb was placed for 10 ft around the pier noses as shown in Figure 3. The upstream protection remained stable near the wing walls where the drawdown occurred, in the approach channel, and around the pier noses for all discharges up to and including 60,000 cfs.

Downstream

22. A combination of riprap (angular stone) and rounded stone protection was placed downstream of the stilling basin as shown in Plate 19. For a distance of 200 ft (sta 0+76.5A to sta 2+76.5A) downstream of the end sill, the channel bottom and side slopes were covered with a 54-in.-thick blanket of protective stone simulating prototype field stone with an average weight of 552 lb. For 100 ft (sta 2+76.5A to sta 3+76.5A), the channel bottom and side slopes were covered with a 42-in.-thick blanket of protective stone simulating prototype field stone with an average weight of 283 lb. For 200 ft (sta 3+76.5A to sta 5+76.5A), the channel bottom and side slopes were covered



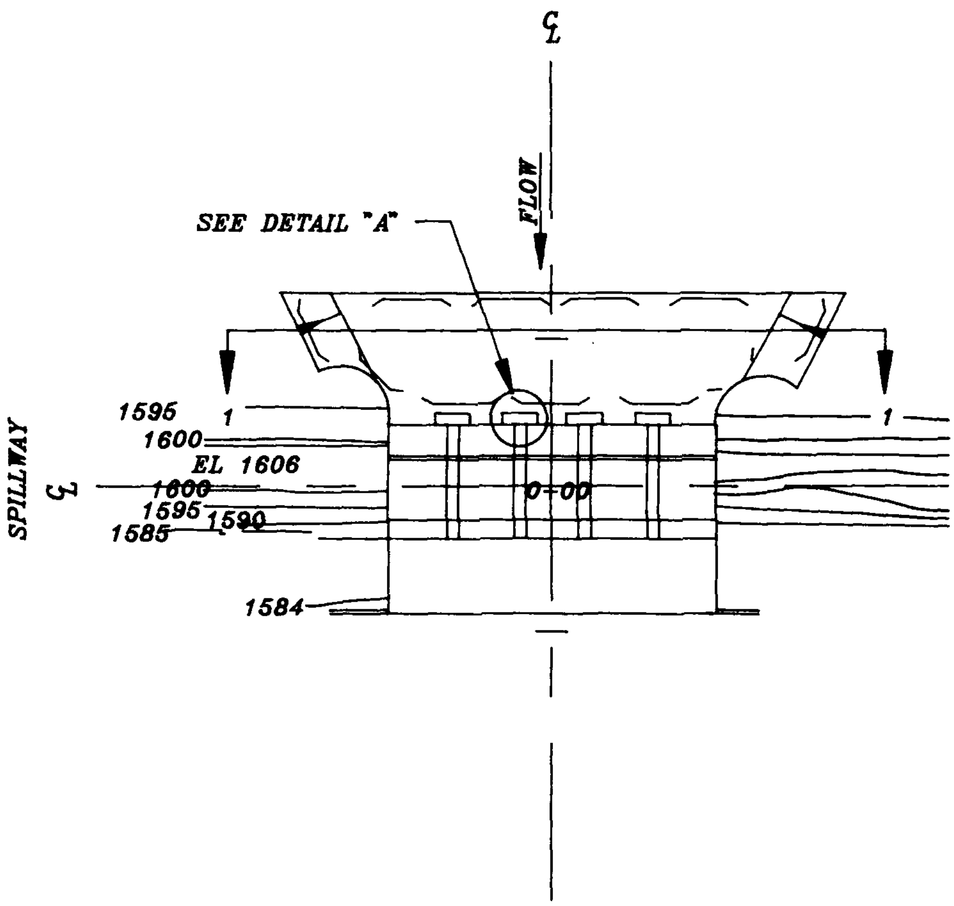


Figure 3. Riprap protection upstream of piers

with a 30-in.-thick blanket of protective stone simulating prototype field stone with an average weight of 119 lb. For 323.5 ft (sta 5+76.5A to the existing channel at sta 9+00A), the channel bottom and side slopes were covered with an 18-in.-thick blanket of protective stone simulating prototype field stone with an average weight of 35 lb. There was some displacement of the smallest stone in the 54-in.-thick blanket, but close inspection revealed that this was minimal. The downstream stone protection remained stable throughout the full range of discharges tested. The duration of each flow tested is given in Table 4.

23. Tests were run with 5,000 cfs and the right gate open 3.2 ft to determine the impact of passing ice over the stone protection. The minimum expected tailwater elevation was set and the upper pool stabilized at el 1599. Ice "pieces" 3-ft square by 2.25-ft thick and 5.4-ft square by 10-in. thick were simulated in these tests. Neither ice size passed under the gate with these conditions. The right gate opening was increased to 7.7 ft and the discharge was increased to 10,000 cfs. The minimum expected tailwater elevation was set and the upper pool stabilized at el 1599. Both sizes of ice passed under the gate. The riprap upstream and downstream of the spillway remained stable during these tests. The duration of each flow tested is presented in Table 4.

PART IV: DISCUSSION AND CONCLUSIONS

24. Performance of the type 3 design approach was generally satisfactory. Localized drawdown around the wing walls did not impact on the hydraulic performance of the structure or damage stone protection.

25. Spillway rating curves for all five gates at full and partial openings were developed for free-flow conditions.

26. During model tests the St. Paul District considered modifying the shape of the pier noses to a shorter type 2 pier nose. Minor surging occurred upstream of the gates with the type 3 pier nose. After observation of the operation of the model, engineers from the St. Paul District decided that modification of the pier nose would create undesirable flow conditions near the gates with and without ice. Therefore, during the model tests, no changes were made to the crest piers.

27. The type 6 design stilling basin consisted of a 51.1-ft-long apron at el 1575 with two rows of 3.5-ft-high baffle piers and a 2-ft-high sloping end sill. Sidewalls were vertical and were 18.0 ft high (top el 1595.0). With the basin raised to el 1575, the hydraulic jump remained in the stilling basin for all discharges up to and including 60,000 cfs at minimum tailwater elevations. Observation of flow conditions in the type 6 design basin revealed a marginal jump in the basin with resulting wave action in the exit area for the 60,000-cfs discharge. The hydraulic performance of the stilling basin was evaluated by conducting hydraulic jump "sweep out" tests to determine the factor of safety of the design for holding the jump in the basin. These tests were conducted by lowering the tailwater below the minimum expected tailwater elevation for 10,000, 30,000, and 60,000 cfs, respectively. When the hydraulic jump began to "sweep out" of the stilling basin, the corresponding tailwater elevation was recorded. WES concluded the hydraulic performance of the type 6 design stilling basin was satisfactory and recommended it for prototype construction.

28. Velocities and center-line water-surface elevations were measured in the approach and exit channels with the minimum expected tailwater elevations in the model for 10,000, 30,000, and 60,000 cfs. Flow patterns and velocities 1 ft above the channel floor and water-surface profiles along the center line of the channel were provided for these flow conditions. Velocities were measured in the exit channel with the minimum expected

tailwater elevations in the model for 5,000 cfs with all gates open 1.7 ft and with one gate open full. Flow patterns and velocities 1 ft above the channel floor were provided.

29. To protect the approach area for discharges up to and including 60,000 cfs, 100 ft (sta 0+59B to sta 1+59B) of protective round field stone with an average weight of 199 lb, then 171 ft (sta 1+59B to sta 3+30B) of protective round field stone with an average weight of 35 lb is required. Protective field stone with an average weight of 283 lb placed 10 ft around the pier noses was required to provide protection for 60,000 cfs. A combination of riprap (angular stone) and rounded field stone protection can be placed to protect the exit channel downstream of the stilling basin. The right wing wall adjacent to the end sill can be eliminated with riprap protection wrapped around the right sidewall. Eliminating the right wing wall had no adverse impact on flow conditions. For protection of the exit channel for discharges up to and including 60,000 cfs, the channel bottom and side slopes should be covered with a 200-ft-long 54-in.-thick blanket of protective riprap with an average weight of 552 lb, a 100-ft-long 42-in.-thick blanket of protective field stone with an average weight of 283 lb, a 200-ft-long 30-in.-thick blanket of protective field stone with an average weight of 119 lb, and a 323.5-ft-long 18-in.-thick blanket of protective field stone with an average weight of 35 lb. The riprap upstream and downstream of the spillway remained stable during tests with flows of 5,000 to 60,000 cfs.

30. With one gate operating, a 7.7-ft gate opening passing 10,000 cfs is required to pass ice 3-ft square by 2.25-ft thick and 5.4-ft square by 10-in. thick. The riprap upstream and downstream of the spillway remained stable during ice passage in the model.

Table 1

Water Surface (WS) Elevations

Gate Opening ft	Q cfs	WS El NGVD	H ft	Condition
Full	8,400	1588.9	4.9	UC
	10,500	1590.4	6.4	UC
	15,500	1591.9	7.9	UC
	32,000	1595.9	11.9	UC
	41,000	1597.9	13.9	UC
	42,000	1598.6	14.6	UC
	49,000	1599.7	15.7	UC
	56,000	1601.4	17.4	UC
	60,000	1601.8	17.8	UC
	62,000	1602.6	18.6	UC

Gate Opening ft	Q cfs	WS El NGVD	H_g ft	Condition
2	5,000	1591.8	8.0	C
	7,800	1597.9	14.1	C
	8,400	1599.5	15.7	C
	9,200	1602.6	18.8	C
3	7,800	1591.2	6.9	C
	10,000	1594.5	10.2	C
	12,000	1598.5	14.2	C
	14,500	1604.6	19.9	C
4	11,000	1592.7	7.9	C
	15,000	1597.8	13.0	C
	17,500	1601.6	16.8	C
	19,000	1604.8	20.0	C
5	15,000	1594.0	8.7	C
	16,750	1594.8	9.5	C
	19,750	1598.5	13.2	C
	20,500	1599.4	14.1	C
10	39,500	1600.5	12.7	C
	46,000	1605.0	17.2	C
	48,500	1607.0	19.2	C

NOTE: C = controlled flow.
UC = uncontrolled flow.

Table 2

Effect of Tailwater on Pool El

Q = 60,000 cfs

Pool El ft	Tailwater El ft
1601.82	1594.5
1601.78	1593.5
1601.75	1592.5
1601.71	1591.5
1601.71	1591.0

Table 3

"Sweep Out" Tailwater Elevations

Q cfs	Pool El ft NGVD	TW El ft NGVD	Gate Opening ft
10,000	1,599.0	1,585.0	1.6
30,000	1,600.0	1,589.0	7.1
60,000	1,601.0	1,591.0	Full

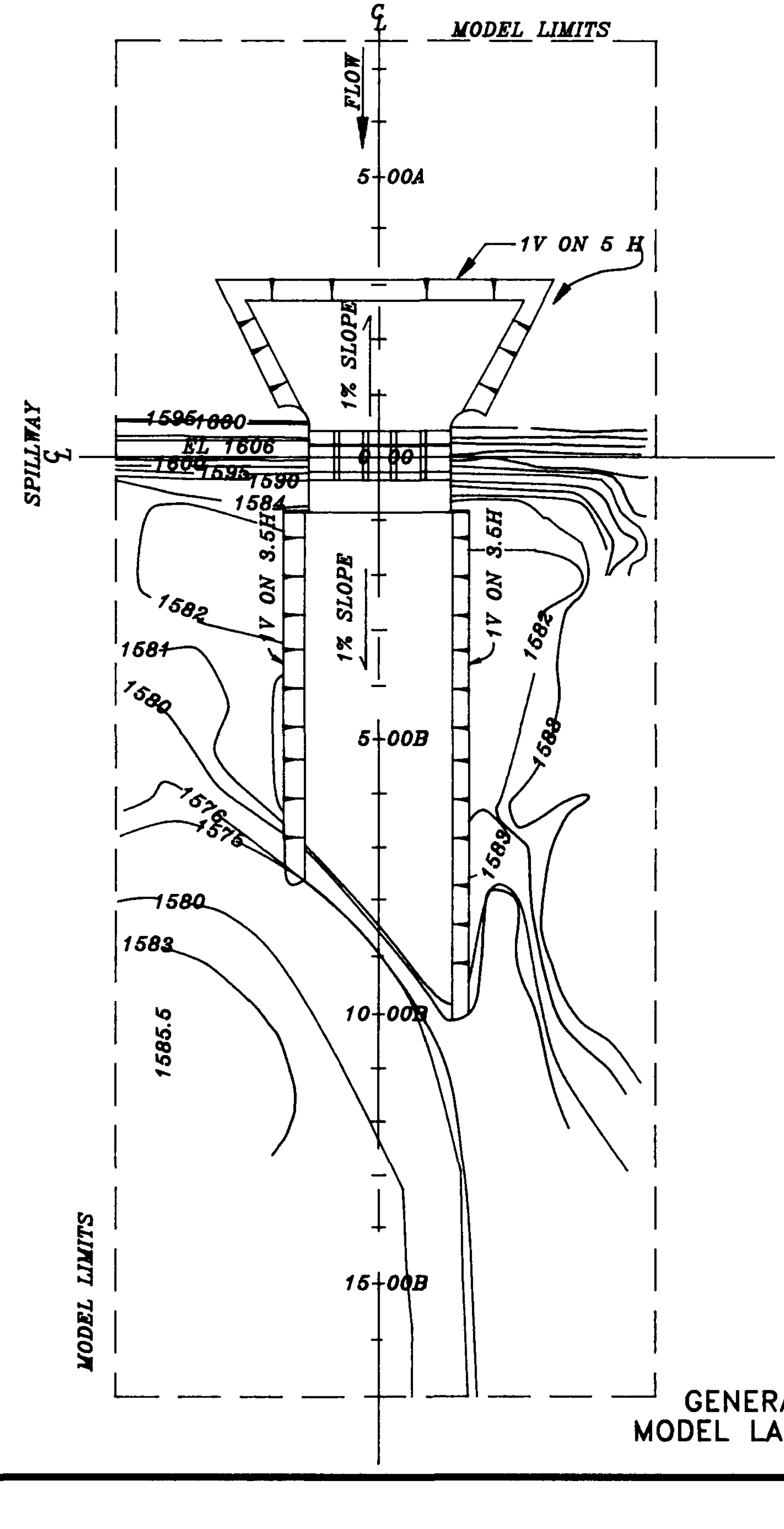
NOTE: Gate openings are referenced to the gate seat at el 1583.4.

Table 4

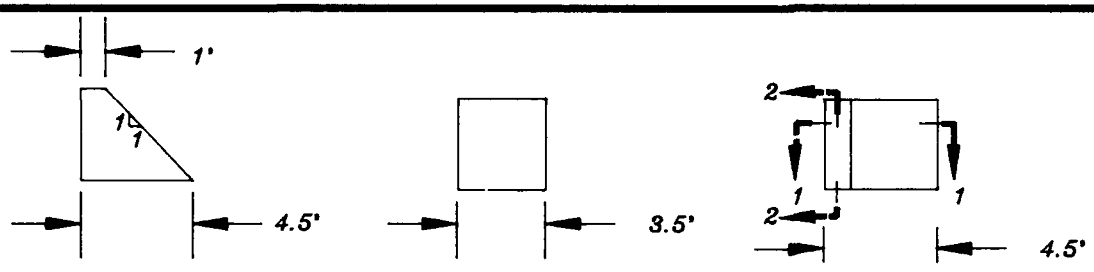
Flow Durations for Riprap Stability Tests

Q cfs	Pool El ft NGVD	TW El ft NGVD	Gate Opening ft	T hr
5,000	1,587.1	1,583.0	1.7	15
5,000	1,591.6	1,583.0	Full*	18
5,000	1,599.0	1,583.0	3.2**	18
10,000	1,594.2	1,586.0	1.7	12
10,000	1,599.0	1,586.0	7.7**	6
30,000	1,600.8	1,590.5	7.7	18
60,000	1,601.0	1,594.5	Full	9

NOTE: Gate openings are referenced to the gate seat at el 1582.8.
* Right gate open.
** Right gate open, passing ice.



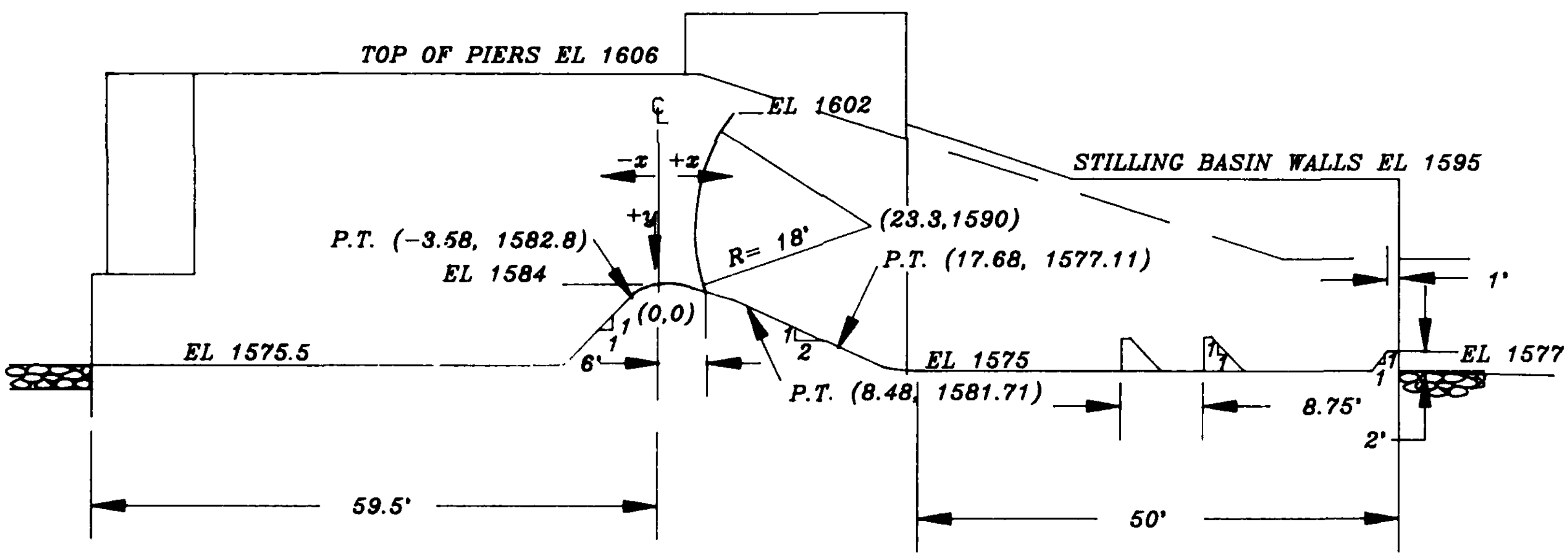
GENERAL
MODEL LAYOUT



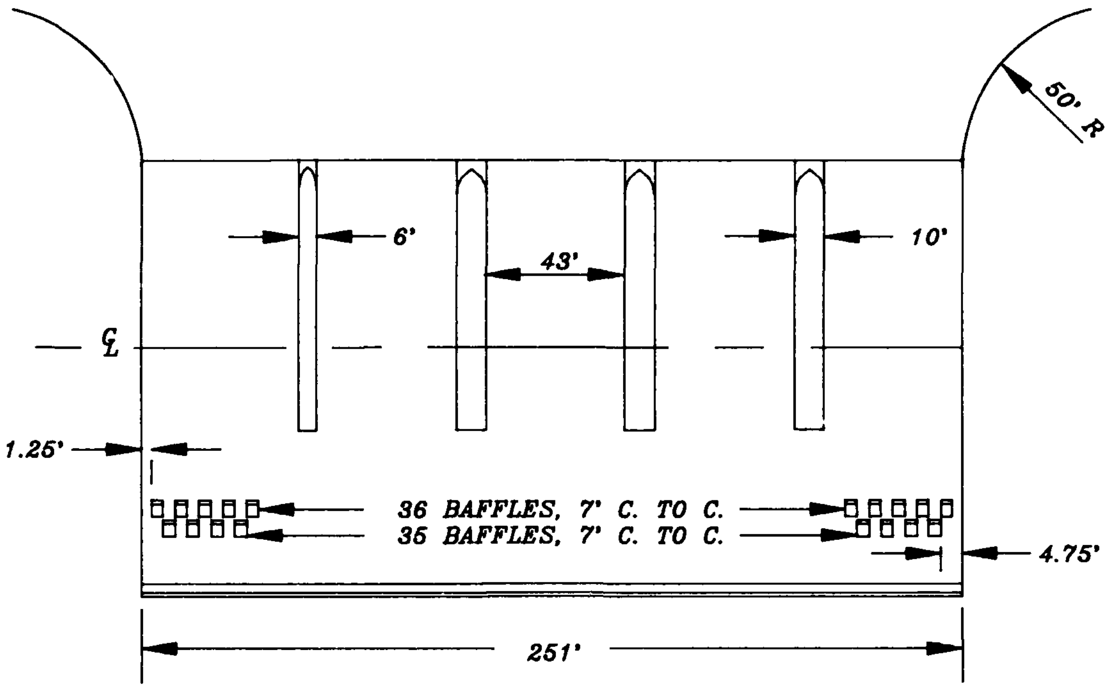
SECTION 1 SECTION 2 TOP VIEW

SPILLWAY CREST COORDINATES

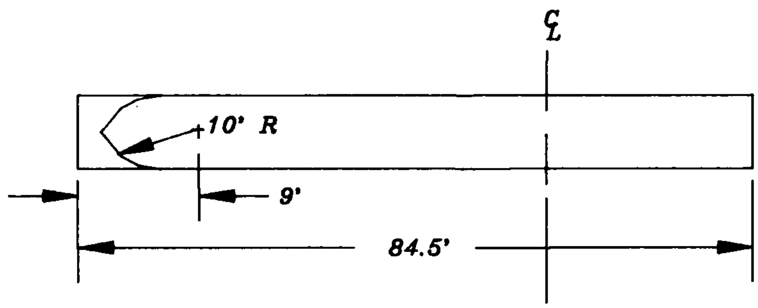
UPSTREAM X	UPSTREAM ELEV (ft)	DOWNSTREAM X	DOWNSTREAM ELEV (ft)
℄ 0	1584.0	℄ 0	1584.0
-0.5	1583.98	1.0	1583.96
-1.0	1583.93	2.0	1583.84
-2.0	1583.70	3.0	1583.67
-3.0	1583.25	4.0	1583.43
P.T. -3.58	1582.80	5.0	1583.14
		7.5	1582.18
		P.T. 8.48	1581.71



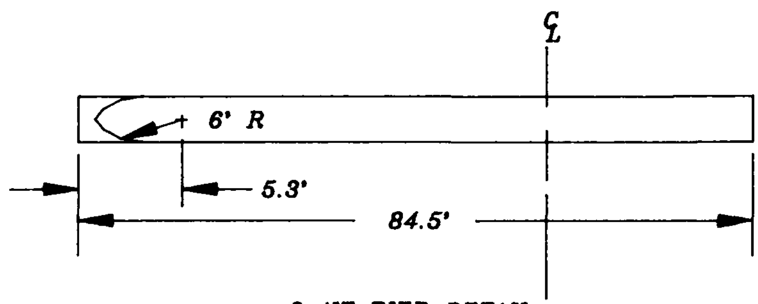
PROFILE
PIER AND SPILLWAY



PLAN

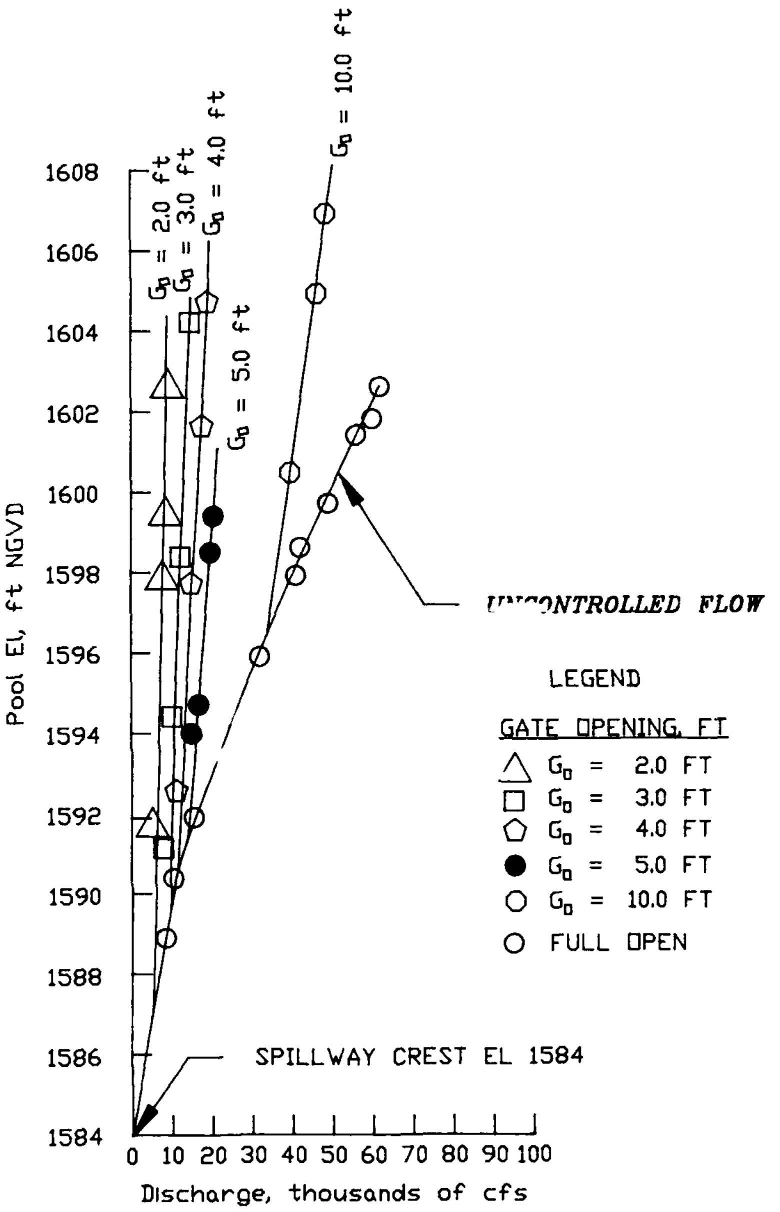


10-FT PIER DETAIL



6-FT PIER DETAIL

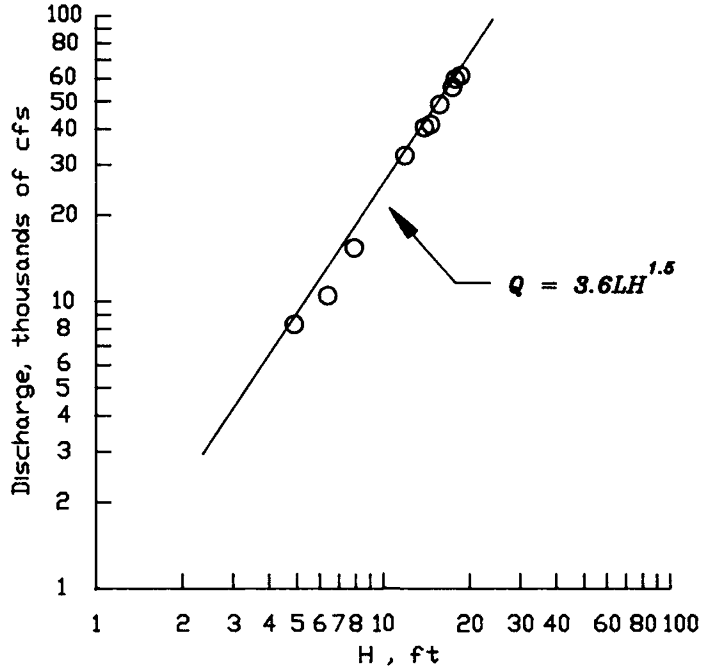
TYPE 6
STILLING BASIN
AND PIER
DETAILS

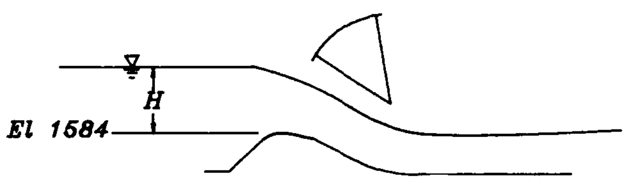


NOTE: GATE OPENING, G_o, IS VERTICAL HEIGHT OF GATE LIP ABOVE GATE SEAT

DISCHARGE RATING CURVE

TYPE 3 DESIGN APPROACH

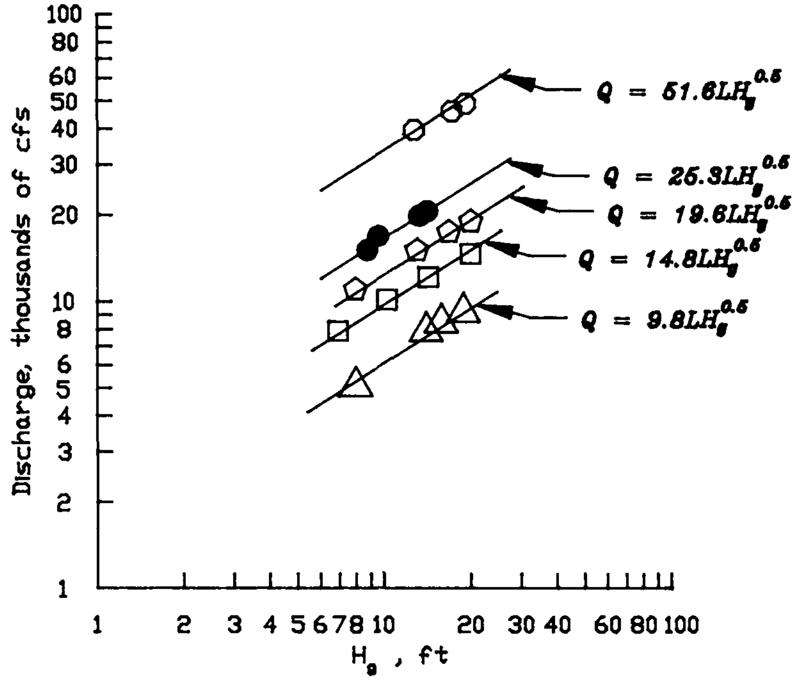




DEFINITION SKETCH

DISCHARGE VERSUS TOTAL HEAD UNCONTROLLED FLOW

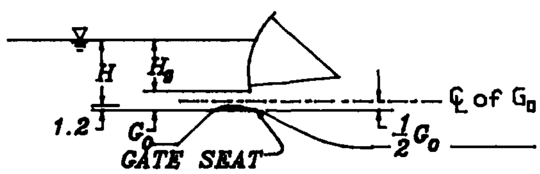
TYPE 3 DESIGN
APPROACH



LEGEND

GATE OPENING, FT

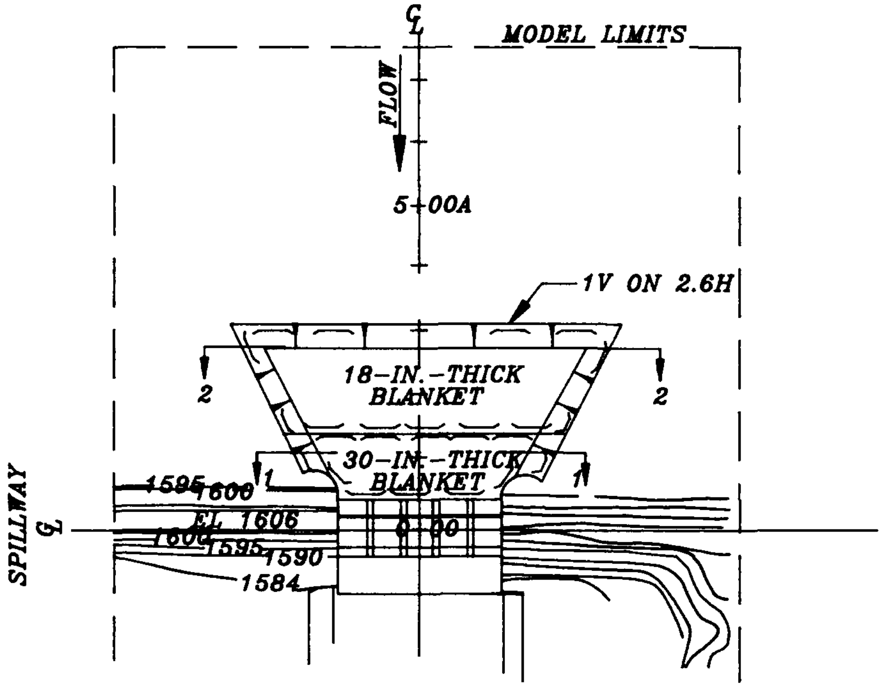
△ G_0 = 2.0 FT
□ G_0 = 3.0 FT
⬠ G_0 = 4.0 FT
● G_0 = 5.0 FT
○ G_0 = 10.0 FT



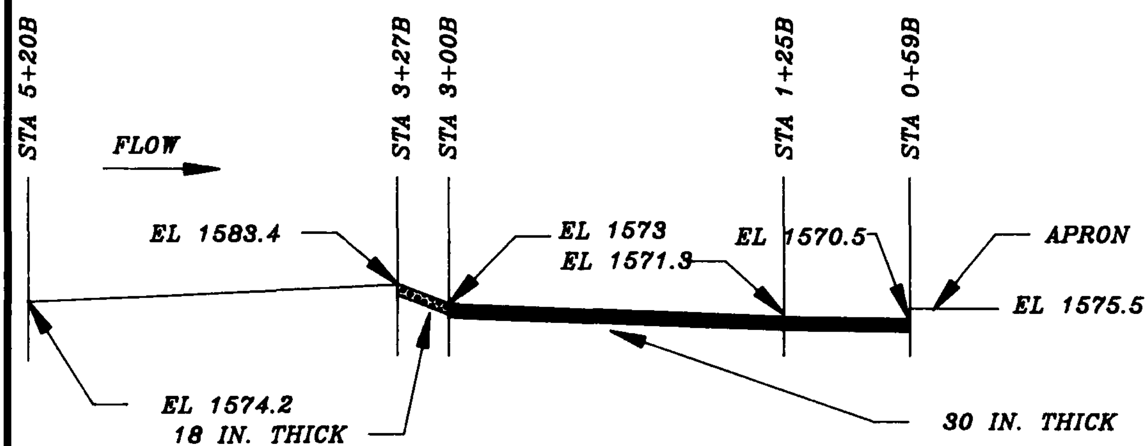
DEFINITION SKETCH

WHERE L = Net crest length, 215 ft
G_0 = Vertical height of gate lip above gate seat
g = acceleration due to gravity, 32.2 ft/sec^2
H_g = head on gate, $(H + 1.2) - \frac{1}{2}G_0$

DISCHARGE VERSUS
HEAD ON GATE
CONTROLLED FLOW
TYPE 3 DESIGN
APPROACH



PLAN VIEW



CENTER-LINE SECTION

RIPRAP DETAIL
UPSTREAM INVERT
LOWERED 5 FT

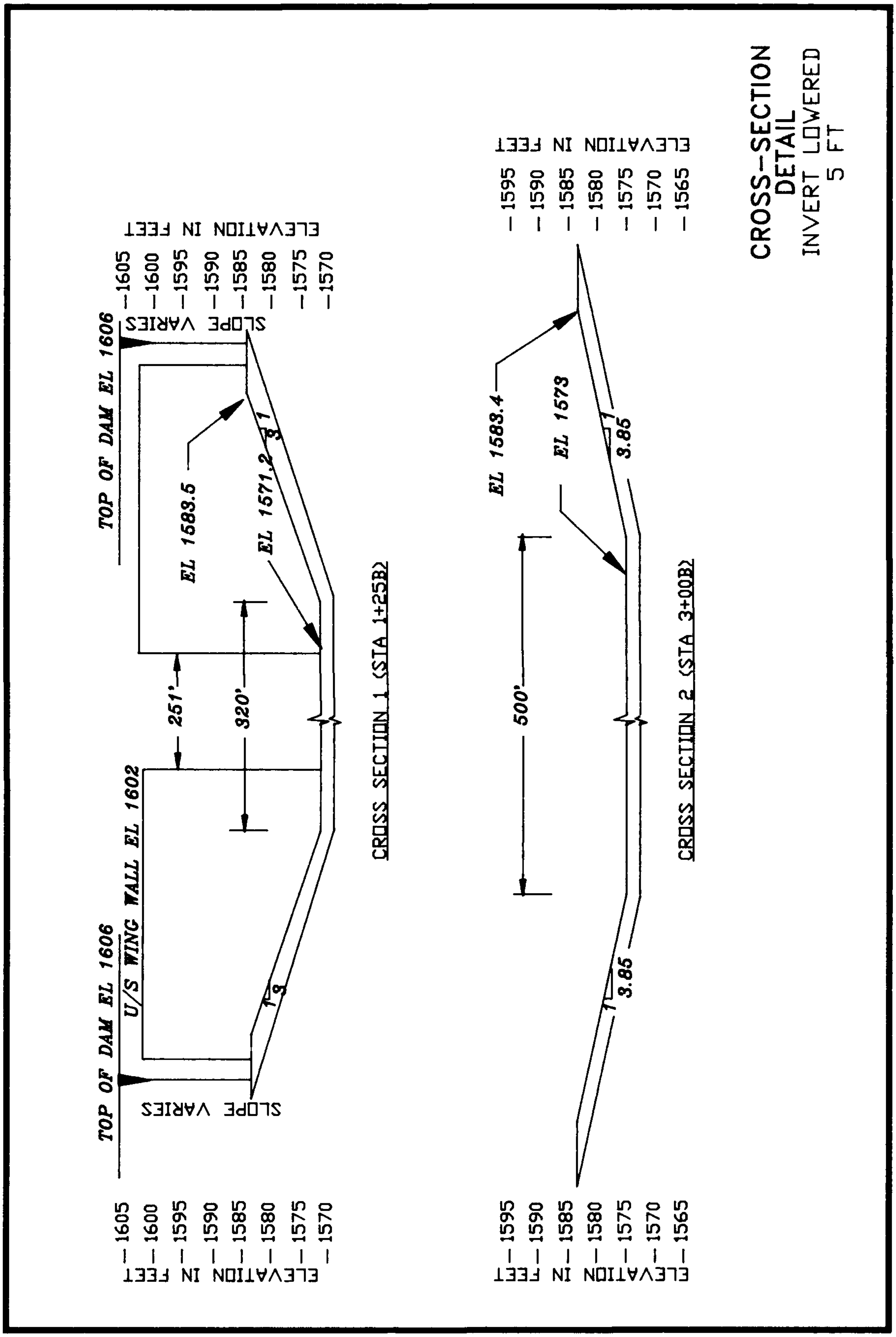
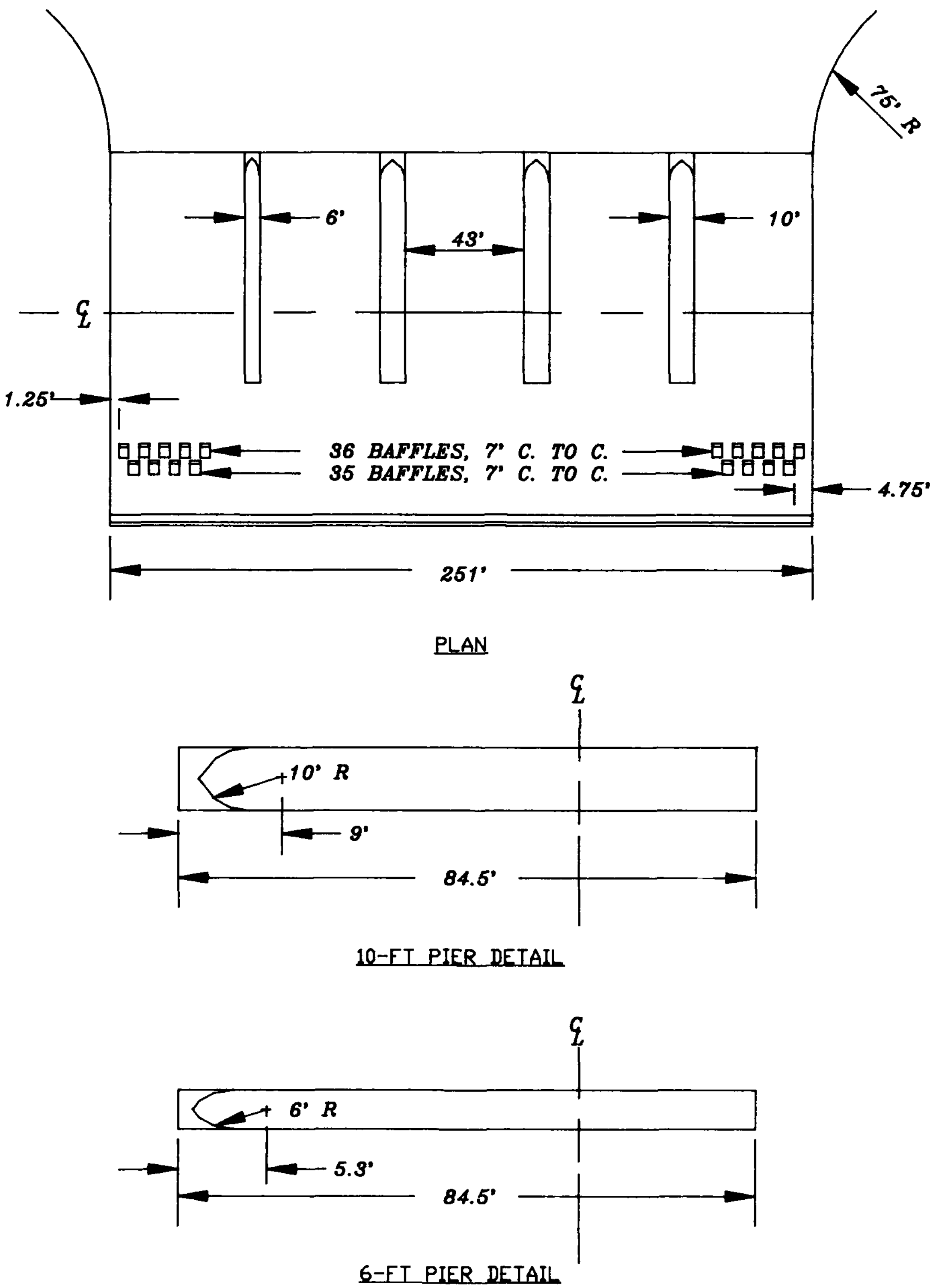
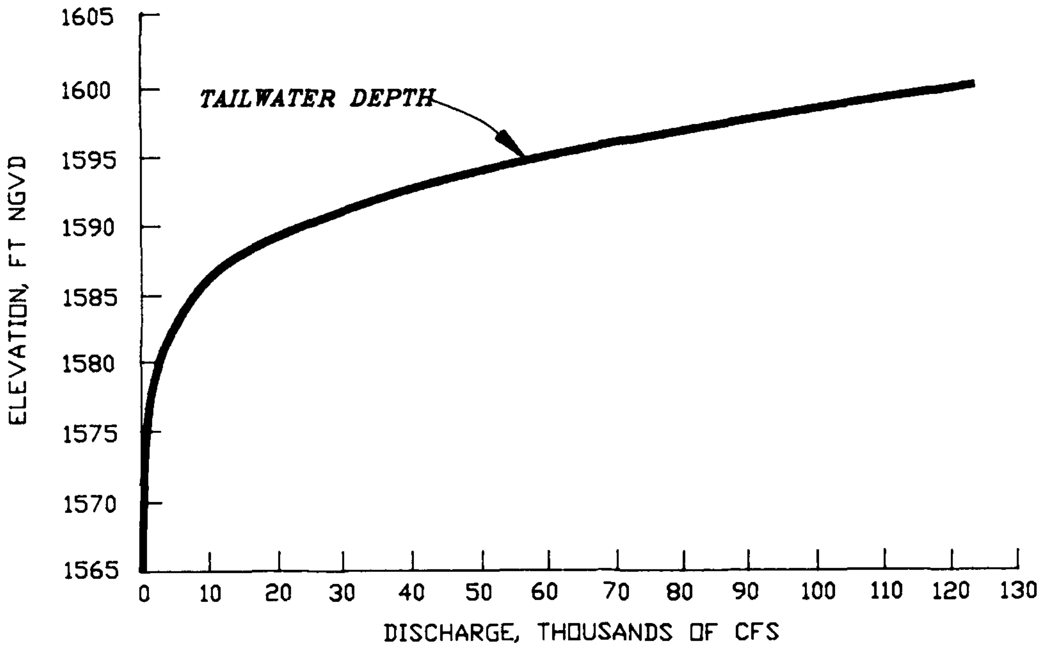


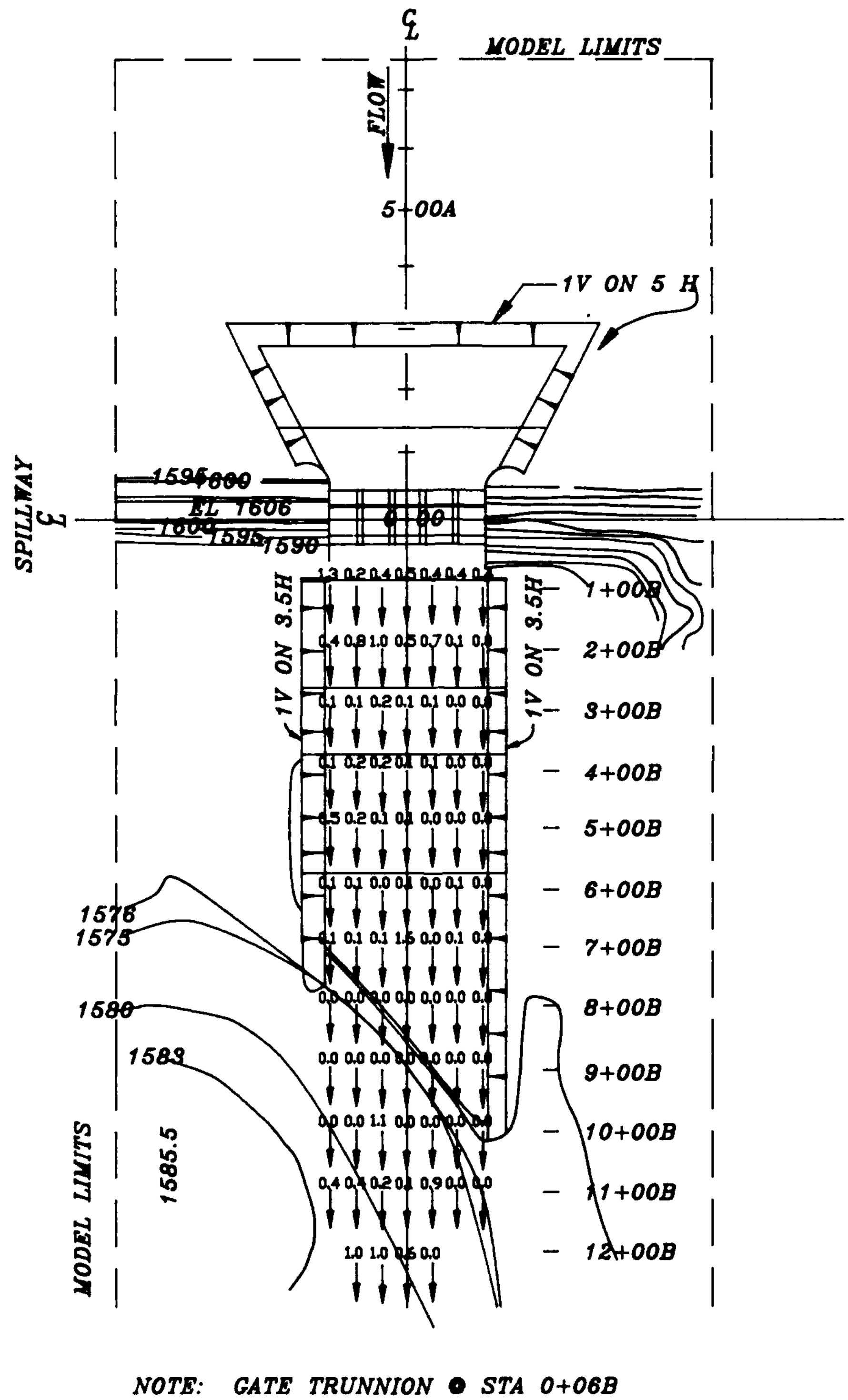
PLATE 7
(SHEET 2 OF 2)



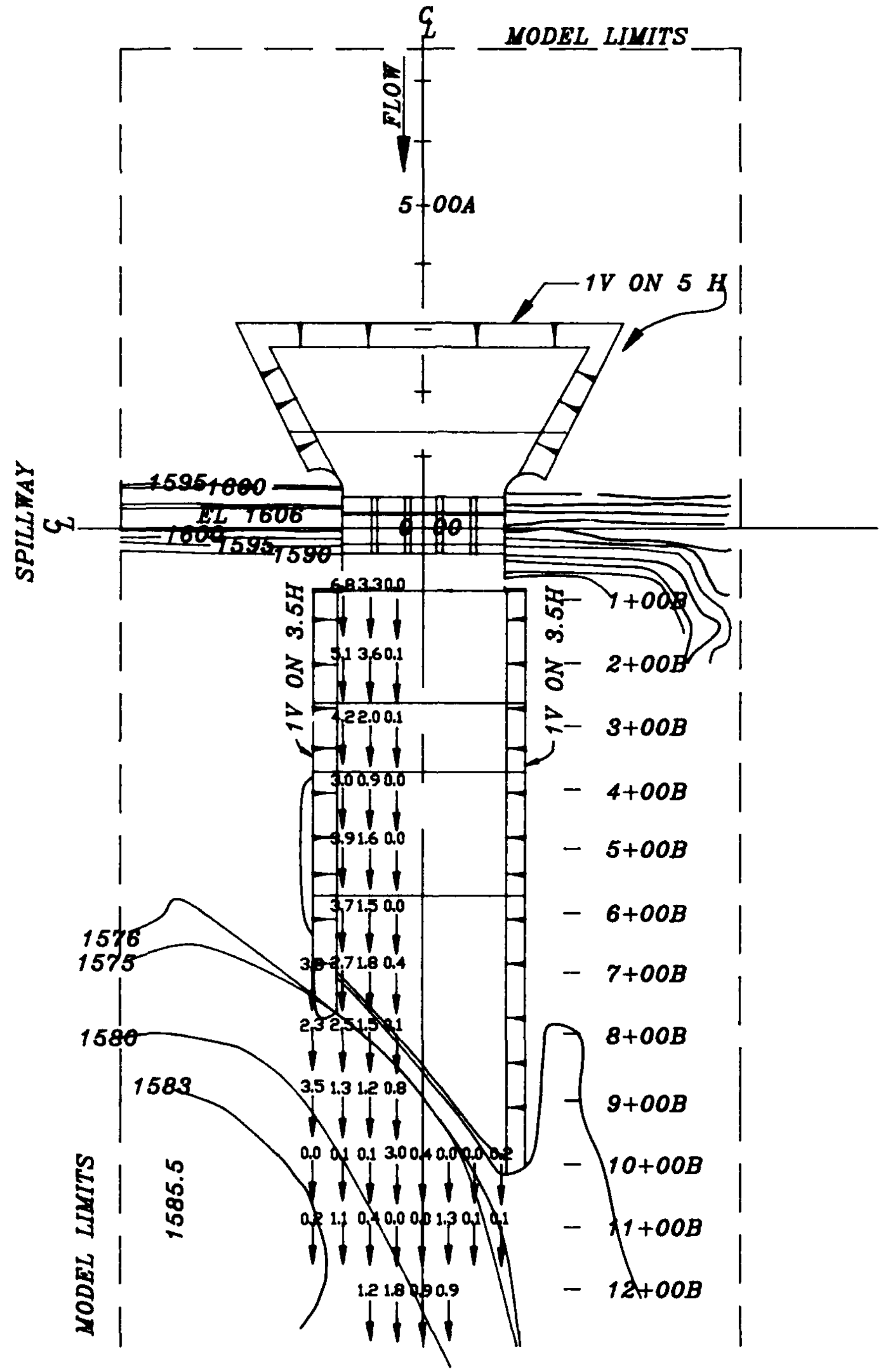
MODIFIED UPSTREAM WING WALL DETAILS



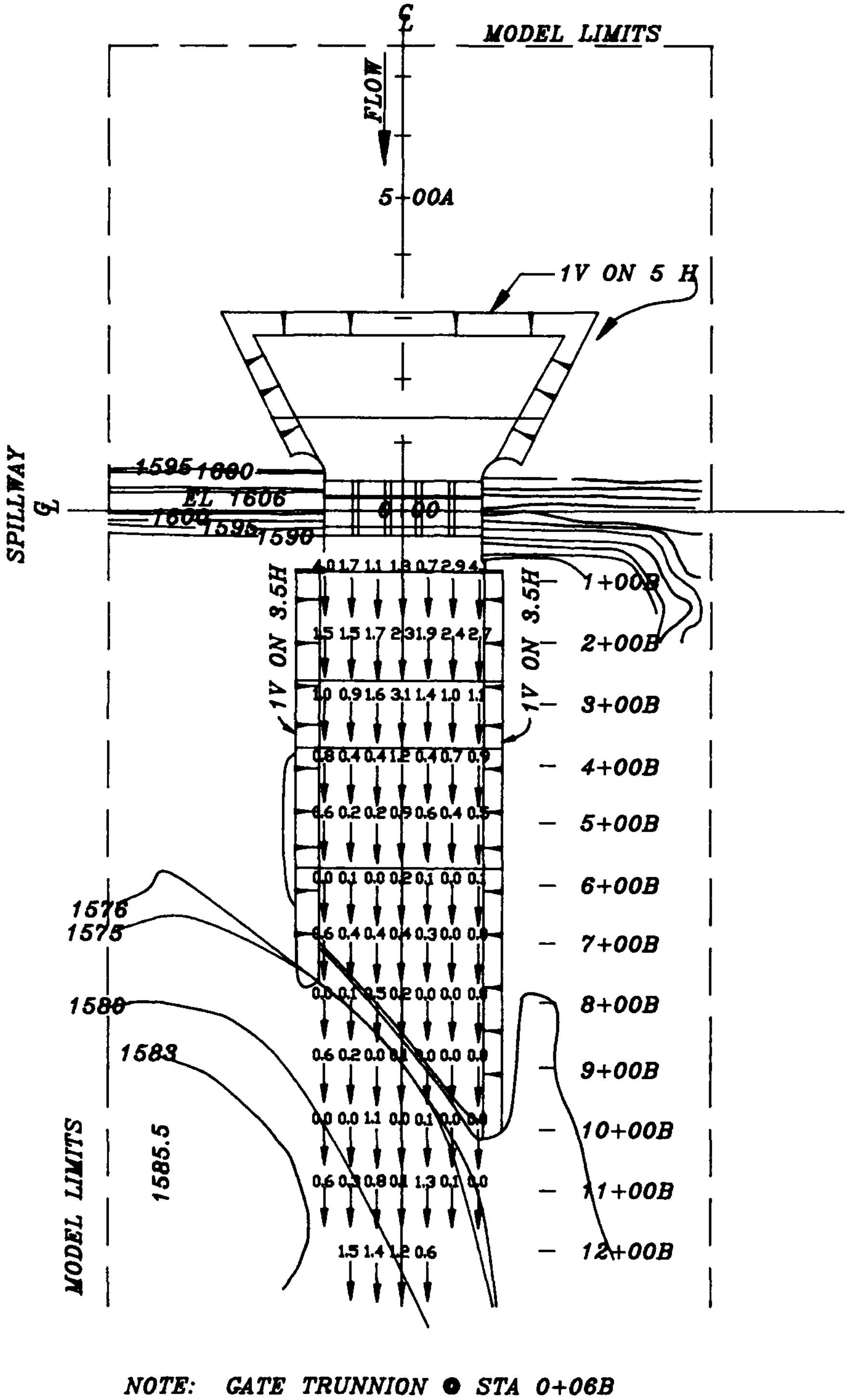
EXPECTED TAILWATER CURVE



BOTTOM VELOCITIES
Q = 5,000 CFS
POOL EL 1587.1
ALL GATES OPEN 1.7 FT

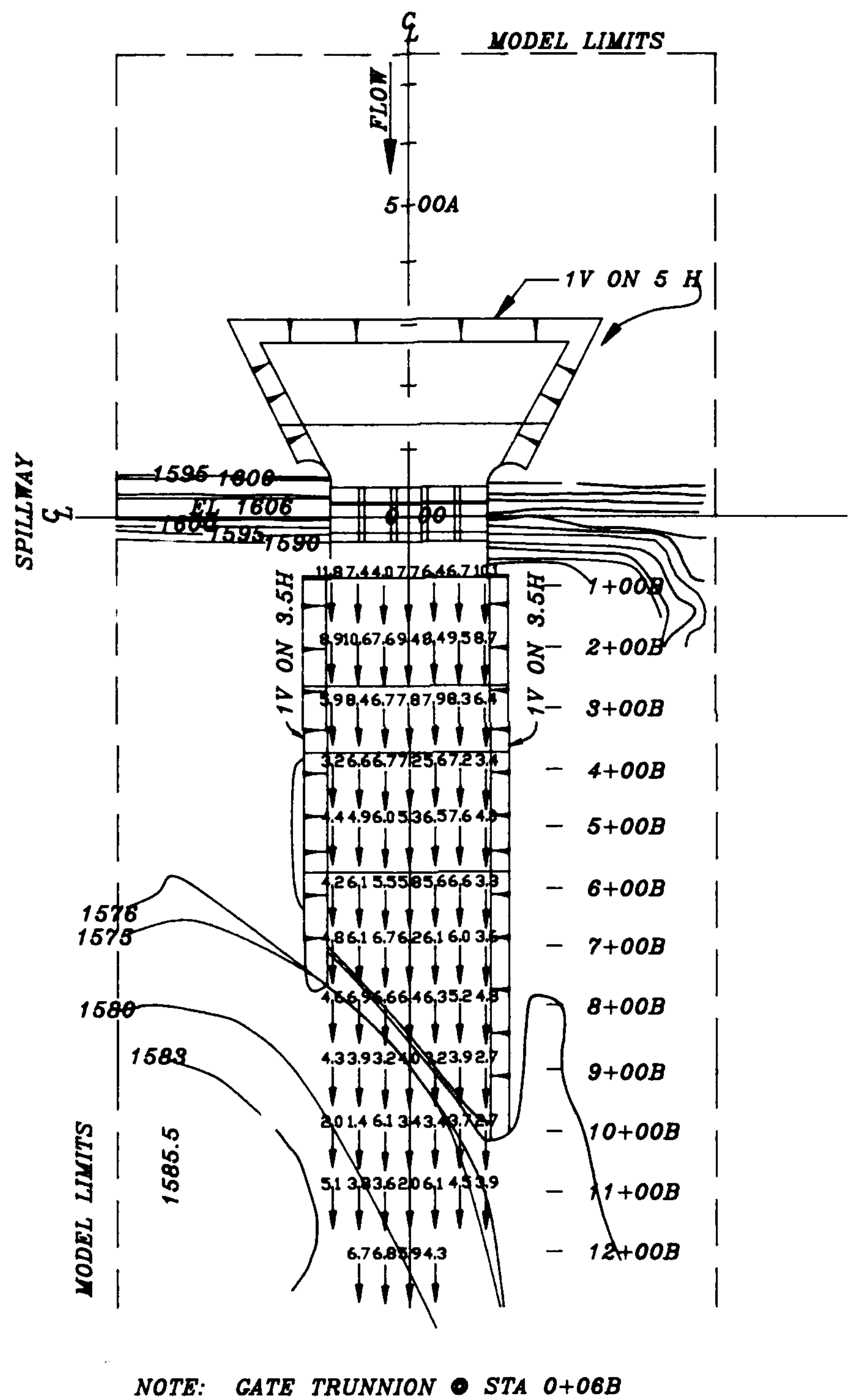


BOTTOM VELOCITIES
Q = 5,000 CFS
POOL EL 1591.6
ONE GATE OPEN FULL

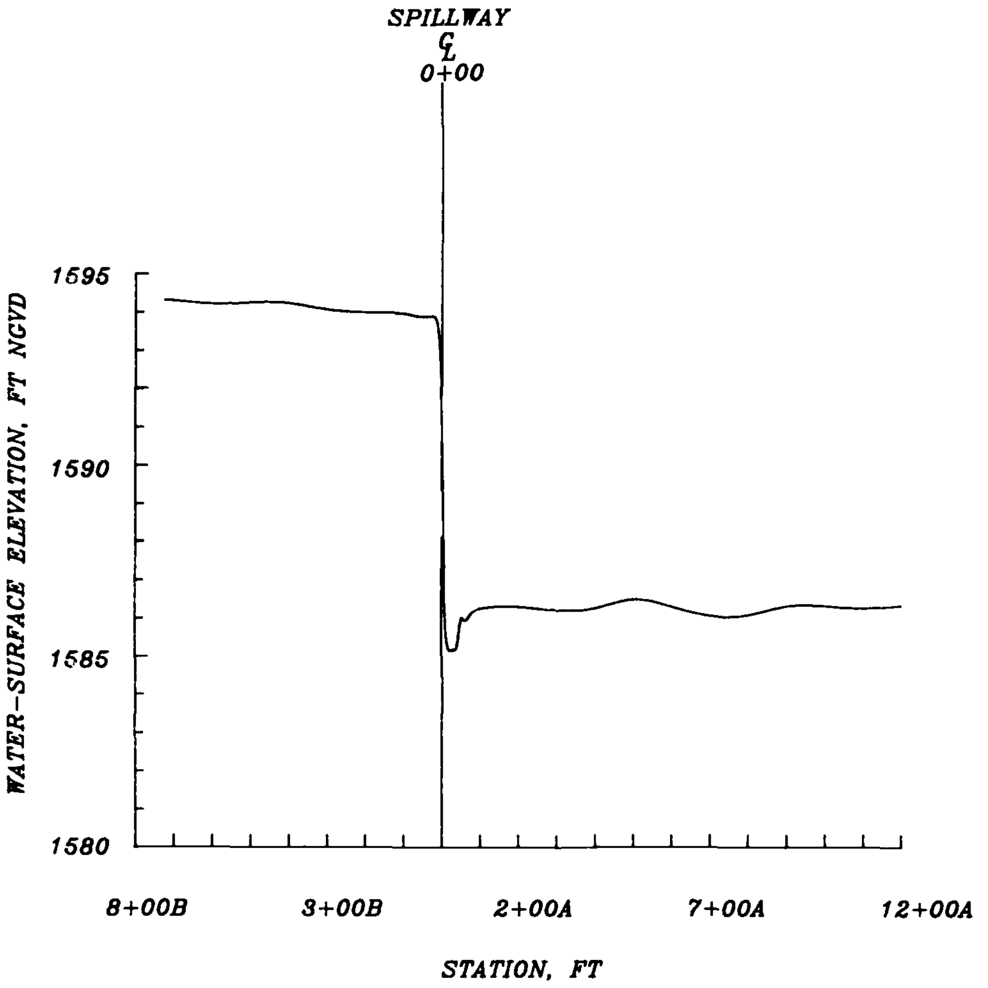


NOTE: GATE TRUNNION ● STA 0+06B

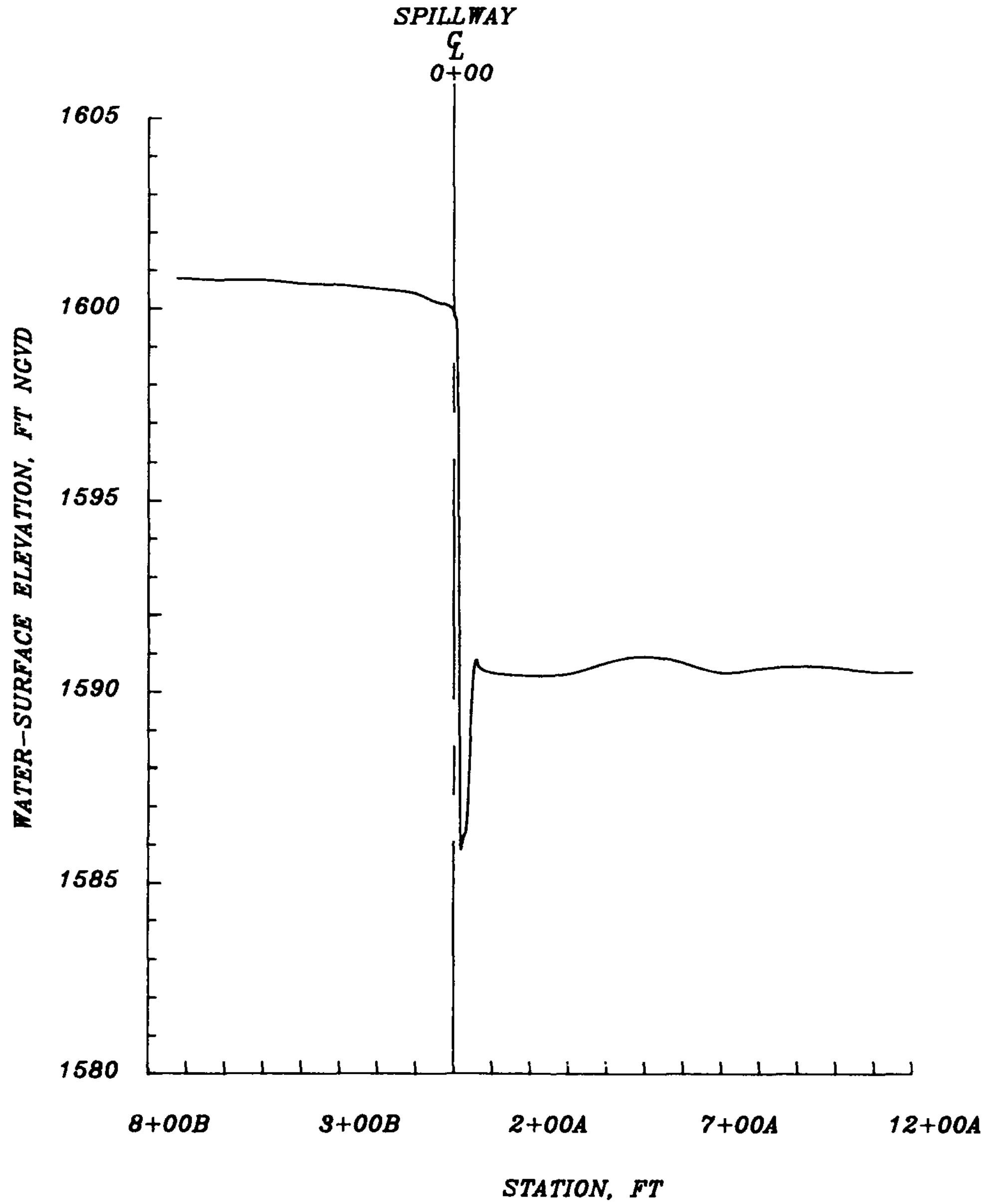
BOTTOM VELOCITIES
Q = 10,000 CFS
POOL EL 1594.2
ALL GATES OPEN 1.7 FT



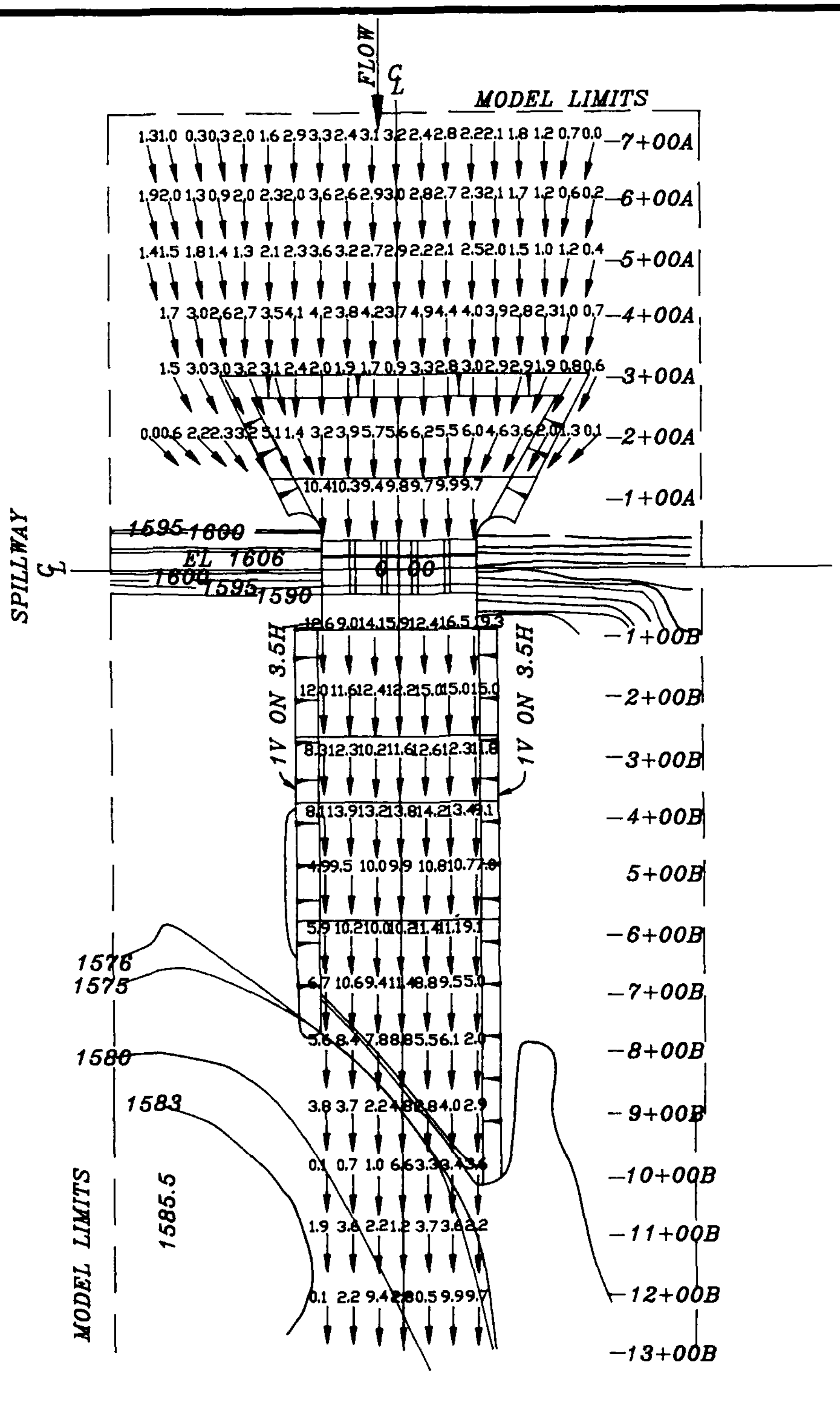
BOTTOM VELOCITIES
Q = 30,000 CFS
POOL EL 1600.8
ALL GATES OPEN 7.7 FT



WATER–SURFACE
CENTER–LINE PROFILE
Q = 10,000 CFS
G_o = 1.7 FT

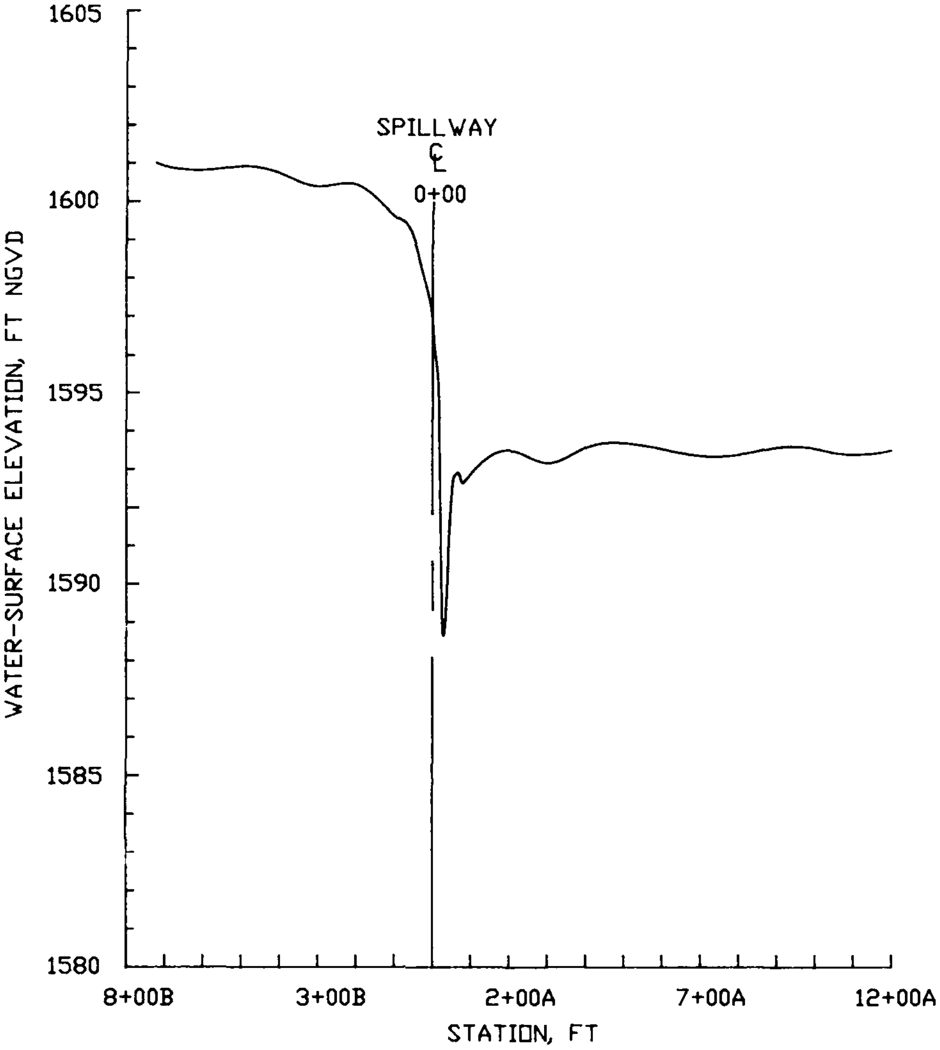


WATER-SURFACE
CENTER-LINE PROFILE
Q = 30,000 CFS
G_o = 7.7 FT

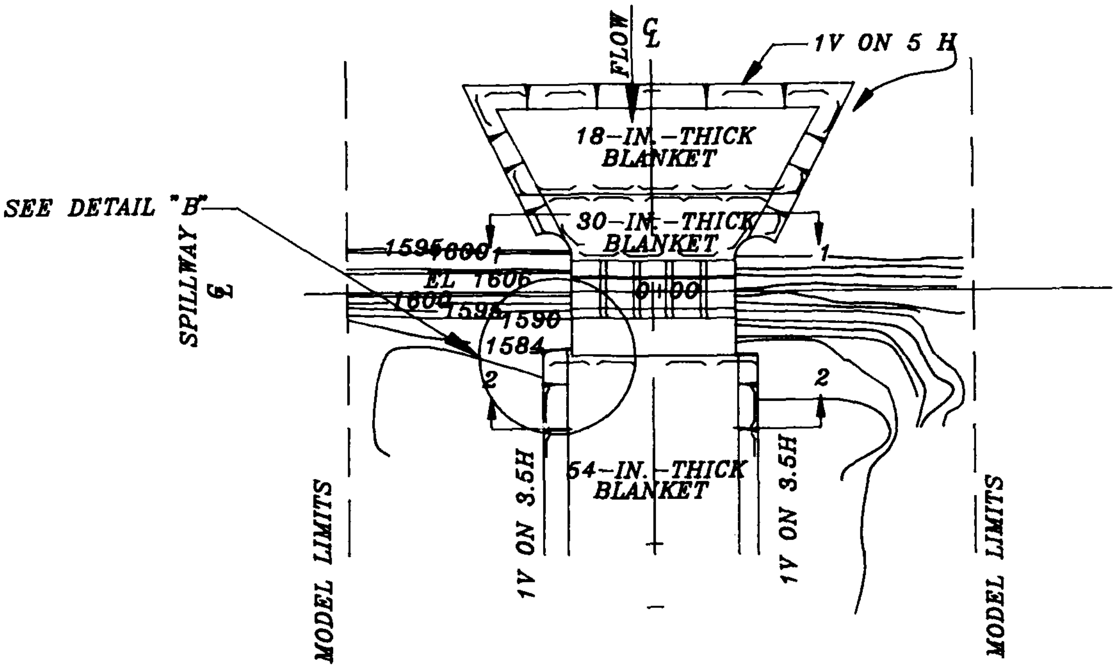


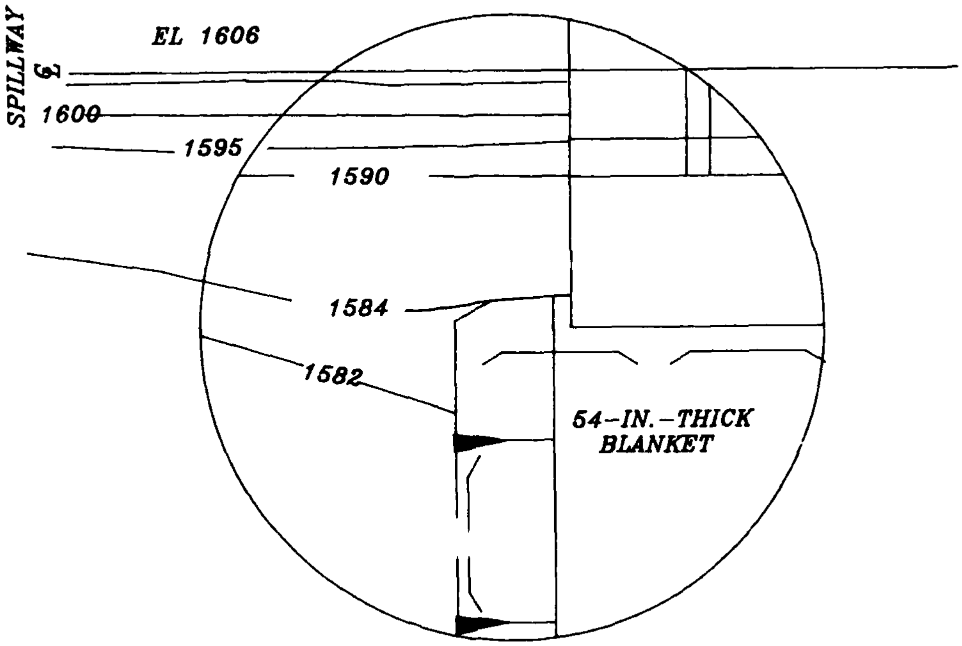
NOTE: GATE TRUNNION @ STA 0+06B

BOTTOM VELOCITIES
Q = 60,000 CFS
POOL EL 1601.8
ALL GATES OPEN FULL



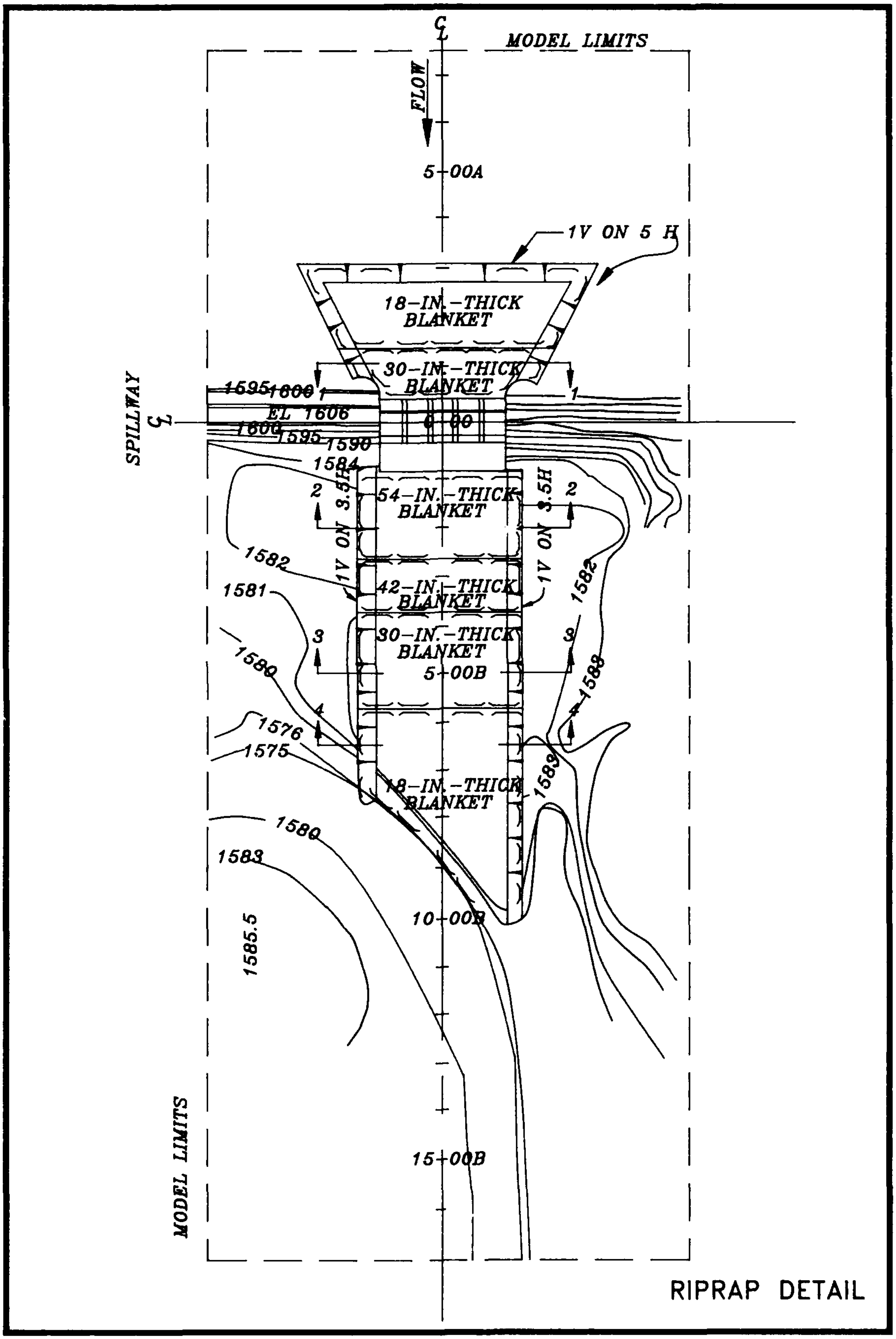
WATER-SURFACE
CENTER-LINE PROFILE
Q = 60,000 CFS
GATE OPEN FULL





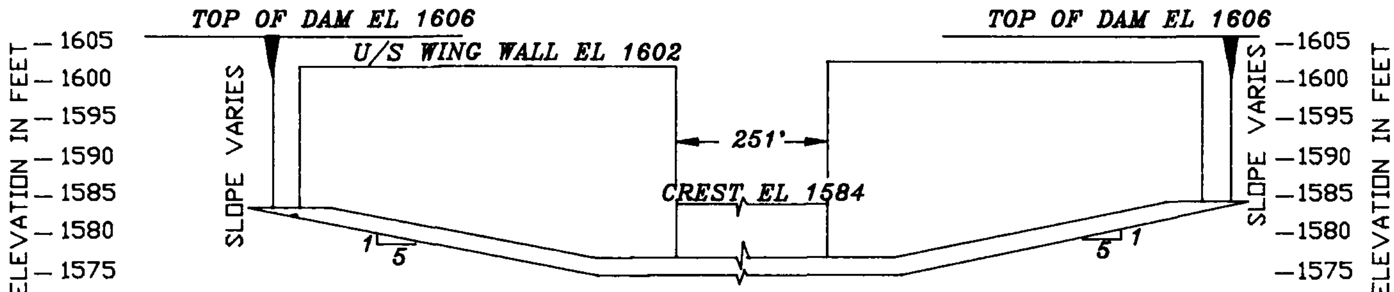
DETAIL 'B'

SIDEWALL
RIPRAP
DETAIL

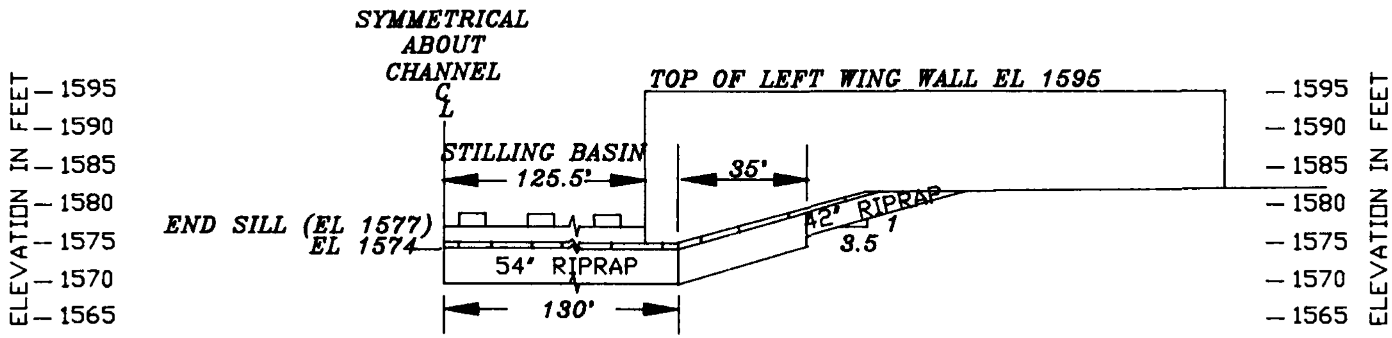


RIPRAP DETAIL

PLATE 19
(SHEET 1 OF 3)

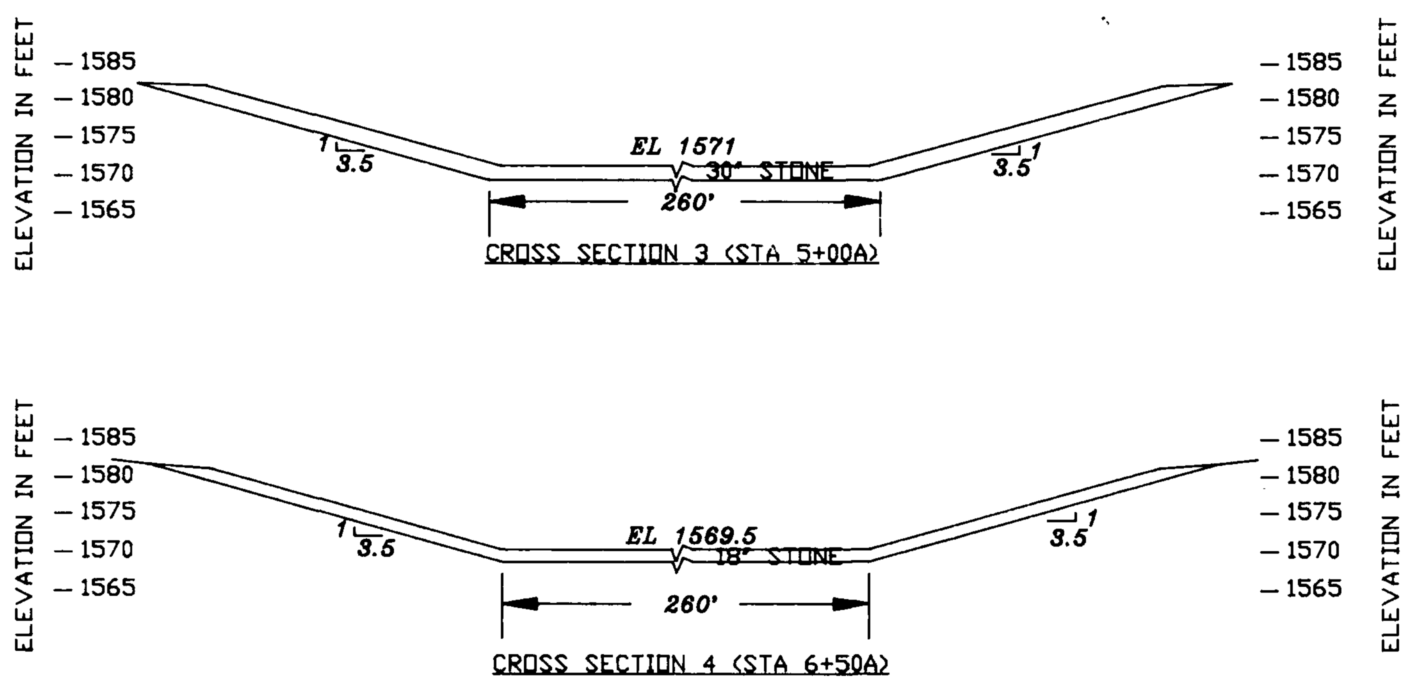


CROSS SECTION 1 (STA 1+25B)

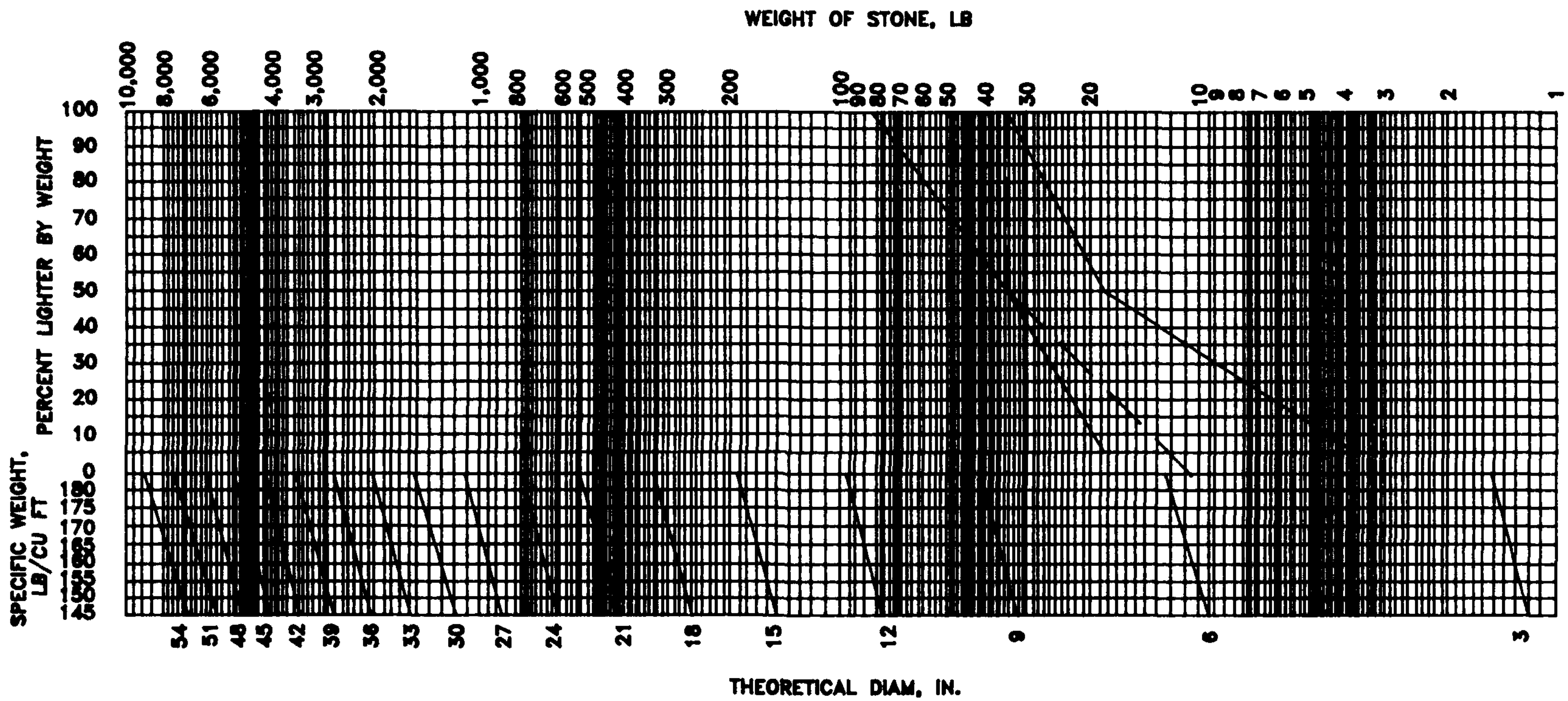


CROSS SECTION 2 (STA 2+00A)

RIPRAP DETAIL
CROSS SECTIONS
1 AND 2

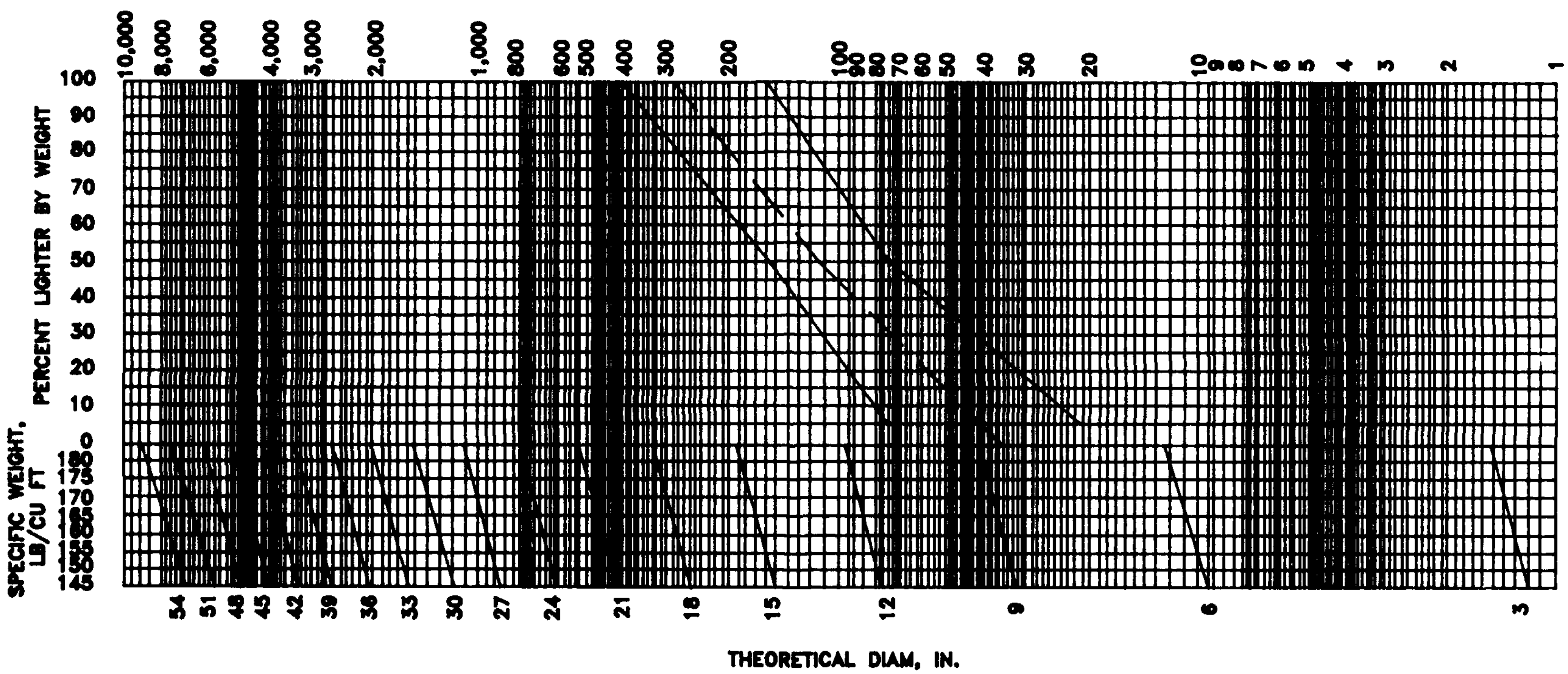


RIPRAP DETAIL
CROSS SECTIONS
3 AND 4



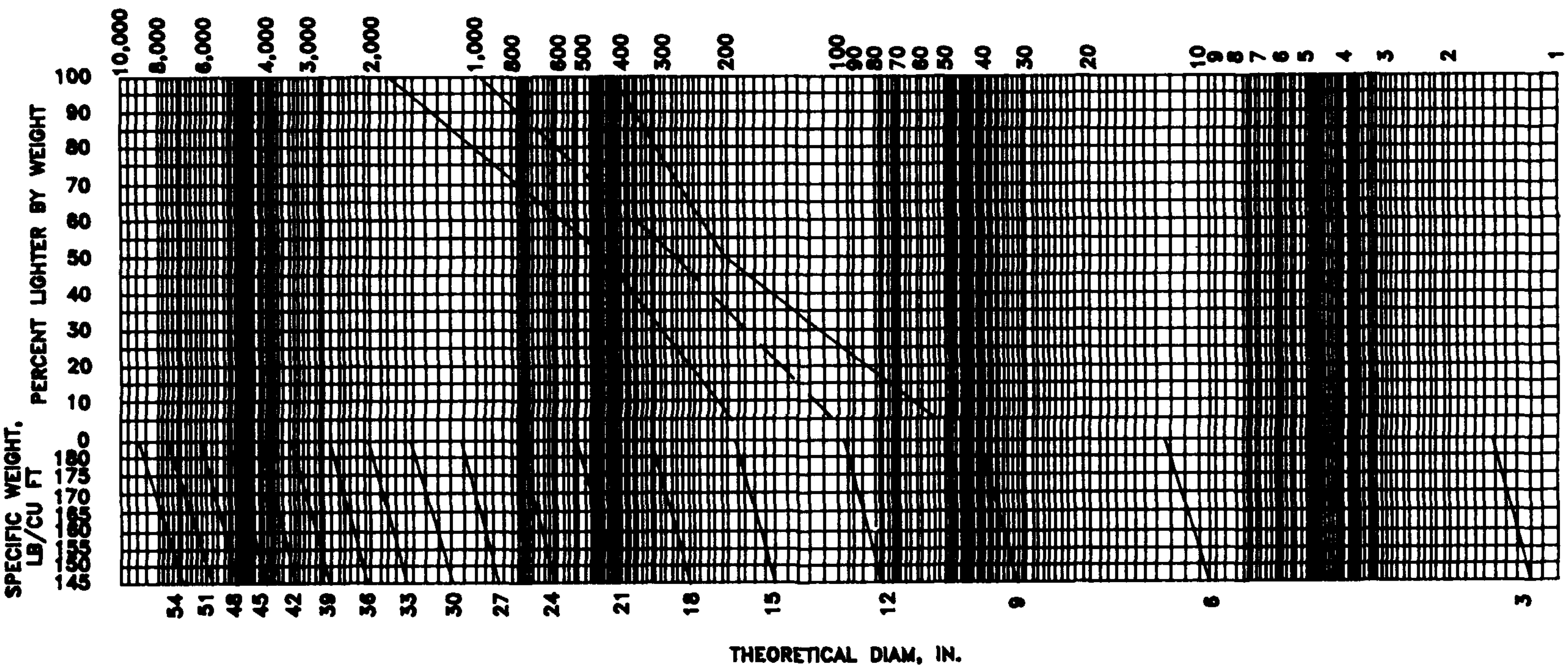
SPECIFIC WEIGHT OF STONE 160 LB/CU FT

STONE GRADATION CURVES
18-IN.-THICK BLANKET



SPECIFIC WEIGHT OF STONE 160 LB/CU FT

STONE GRADATION CURVES
30-IN.-THICK BLANKET



SPECIFIC WEIGHT OF STONE 160 LB/CU FT

STONE GRADATION CURVES
42-IN.-THICK BLANKET

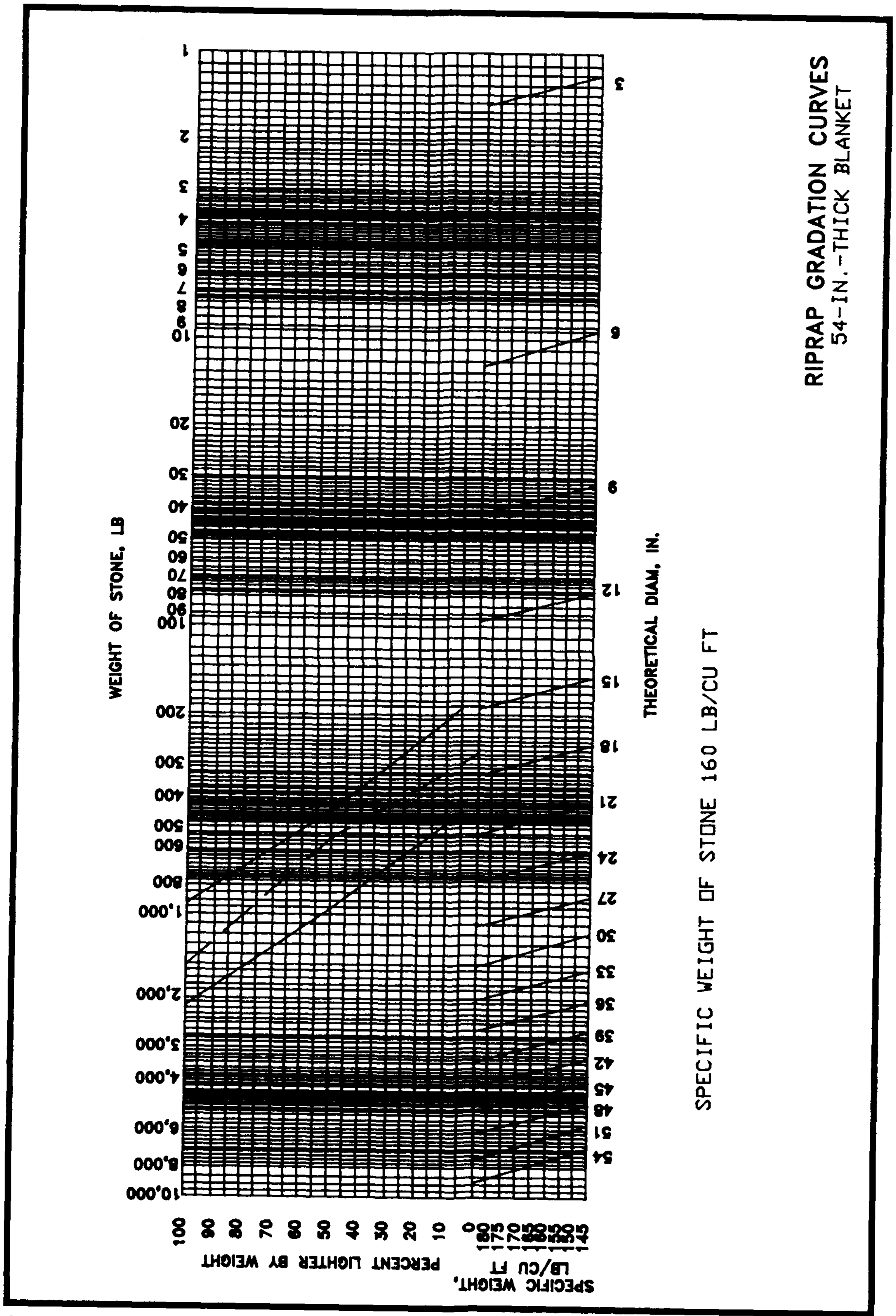

RIPRAP GRADATION CURVES
54-IN.-THICK BLANKET
SPECIFIC WEIGHT OF STONE 160 LB/CU FT
WEIGHT OF STONE, LB
THEORETICAL DIAM, IN.
PERCENT LIGHTER BY WEIGHT
SPECIFIC WEIGHT, LB/CU FT

PLATE 23

Waterways Experiment Station Cataloging-in-Publication Data

Cooper, Deborah R.
Lake Darling spillway, Souris River, North Dakota. Report 2, Modified spillway : hydraulic model investigation / by Deborah R. Cooper ; prepared for US Army Engineer District, St. Paul.
48 p. : ill. ; 28 cm. — (Technical report ; HL-88-9 rept. 2)
1. Spillways — North Dakota — Darling, Lake — Design and construction. 2. Diversion structures (Hydraulic engineering) — Souris River — Models. 3. Hydraulic models. 4. Stilling basins — Evaluation. I. United States. Army. Corps of Engineers. St. Paul District. II. U.S. Army Engineer Waterways Experiment Station. III. Title. IV. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-88-9 rept. 2.
TA7 W34 no.HL-88-9 rept.2